ENGINEERING NEWS

A JOURNAL OF CIVIL, MECHANICAL, MINING AND ELECTRICAL ENGINEERING

Vol. LVI. No. 11. 220 Broadway, New York, September 13, 1906.

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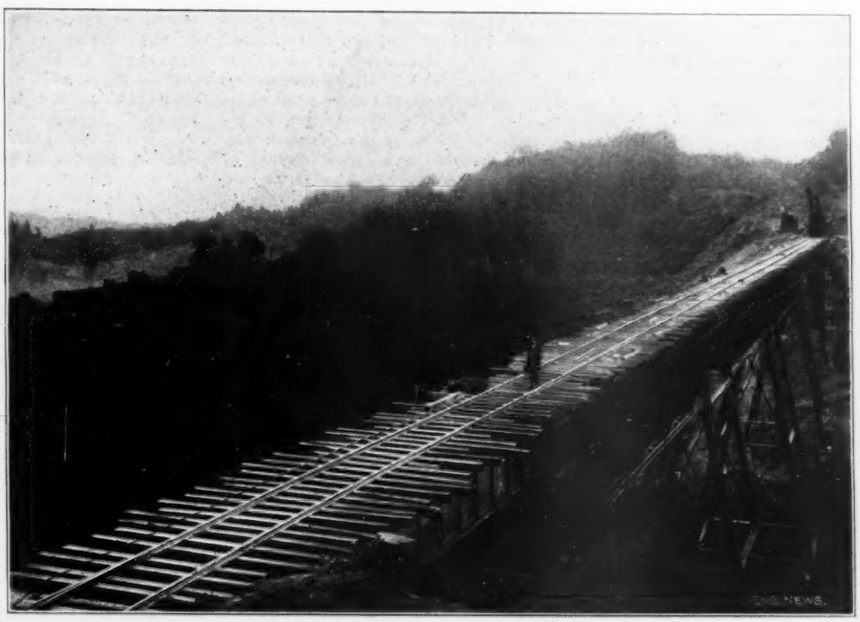
THE CANEADEA VIADUCT OF THE BUFFALO & SUSQUEHANNA RY.

By Joseph Mayer,* M. Am. Soc. C. E.

Among the steel structures of the new Buffalo & Susquehanna Ry., which extends from Wellsville, N. Y., on the Pennsylvania State line to Buffalo, N. Y., the most noteworthy is a high steel viaduct on which the line crosses a deep and narrow gorge near Caneadea, N. Y. The viaduct is 754 ft. long between abutments, and the roadbed (base of rail) is 175 ft. above water level in the stream. The two highest towers are 147 ft. high from top of pier to bottom chord of truss, corresponding to a height of 167 ft. to base of rail. The viaduct is now in process of erection. The elevation and plan shown in Fig. 1, herewith, give the general dimensions of the structure and exhibit its disposition of parts.

The viaduct was designed with special view to facilitating erection by derrick-car and without falsework. For this purpose, 100 ft. was made the maximum span. The two highest towers are 40 ft. wide, the others 30 ft. wide, in the direction of the axis of the viaduct. The latter, therefore, consists of 30-ft. and 40-ft. tower spans, which are plate-girders, alternating with plate-girder and truss spans 30 to 100 ft. long. To equalize the longitudinal forces acting on the two highest towers, each of them is made to carry one fixed and one rolling end of a 100-ft. span.



VIEW ALONG DECK OF CANEADEA VIADUCT AFTER COMPLETION OF STEELWORK.

The derrick car used to erect the viaduct weighs about 30 tons and is provided with a steam engine for its own motion by means of a chain gear to a front axle. The car has two trucks, the front truck having the strength of that of a 100,000-lb. capacity car. It carries an independent steam hoisting engine, a boiler and a large water tank with capacity for one day's work. Near the front of the car is the pivot of a 60-ft. boom which can be swung through about half a circle and lowered to nearly a horizontal position. At the four corners of the car, clamps are provided to anchor it down to the rails or to the stringers or top chords of the deck spans of the viaduct. Along the sides are large boxes for storing the necessary erection tools. In the center is a tunnel for storing the boom during shipment. The latter has rollers on three sides to facilitate sliding it into the tunnel. The car can be made ready for work, or, if in working order, ready for shipment, within 1½ hrs. It will move by its own power at the rate of about 10 to 12 miles per hour. Its lifting capacity is about 25 tons when the end of the boom is between the rails.

The erection was started from the south or Wellsville end, the material having previously been unloaded along the track near this end. As already stated, no falsework was used, except some platforms on the bottom of the gorge to give a level working floor. The standard track up to the end of the viaduct was built before the arrival of the material, and was extended over the viaduct as the erection proceeded. The erection car moving on this track picks up the material by means of block and fall suspended from the end of the boom, and moves to the end of the track, where the material is lowered and put in position to build a tower. The connections are first bolted and afterwards riveted. After a tower is built the deck plate-girder spans to and above it are placed and connected up, the track extended and the process repeated with the following towers.

For the erection of the 100-ft. truss spans the material for the trusses is first lowered to the ground immediately below the final position of the span, or if the span is above a steep slope it is lowered to an erection platform previously built. The trusses are then connected up and raised, one at a time, to their final positions. The laterals and the floor are then placed and the track extended.

This method of erection for high viaducts is now a rather common one. With short spans it is much cheaper than the use of falsework. The erection car is a permanent tool which can be used for many viaducts in succession. It is a contrivance giving a great advantage to a large company, which alone can keep such a car reasonably busy.

Riveted 100-ft. spans can be safely managed in this manner; longer spans are very difficult and risky to handle. It is therefore advantageous to limit the length of the spans to about 100 ft., even where mere weight economy would dictate longer spans.

The details of the posts and trusses may in general be passed over with the comment that a stiff riveted type of construction is carried out in all parts. The columns in all cases have horizontal caps, on which the trusses or plate-girders are supported by exactly the same means as would be used on masonry abutments. Each truss or girder connecting succeeding towers has an expansion bearing at one end. The caps of the columns in any one tower are in a horizontal plane. All girders and trusses, however, are set parallel to the grade line; in the case of the tower spans, the rise of about 2¼ ins. is made up by the extra thickness of the bed-plates at the higher end.

The connection of adjacent spans of different depths offers some features of interest. The fixed end of a tower span is riveted to the fixed-end post of the appropriate adjacent truss, and at its other end rests on a bracket built on the moving-end post of the other truss, the tower girder thus has a sliding expansion connection with the latter, not, however, to allow for expansion of the tower girder, but to permit the truss and bracket to move under it with the expansion movement of the truss. An exception to this arrangement occurs at post N, where the spacing of the girders narrows to 7 ft. while the trusses are 9 ft. on centers. Here the end frame of the truss has a transverse plate-girder, as shown in Fig. 2, and the 30-ft. tower girders are riveted to the web of this cross-girder, although this is the expansion end of the truss. This virtually makes the tower span a part of the 100-ft. truss, as concerns expansion, and necessitates an expansion bearing for the tower girder at post P. The elevation, Fig. 3, and the detail plan in Fig. 4 show how such a bearing is provided.

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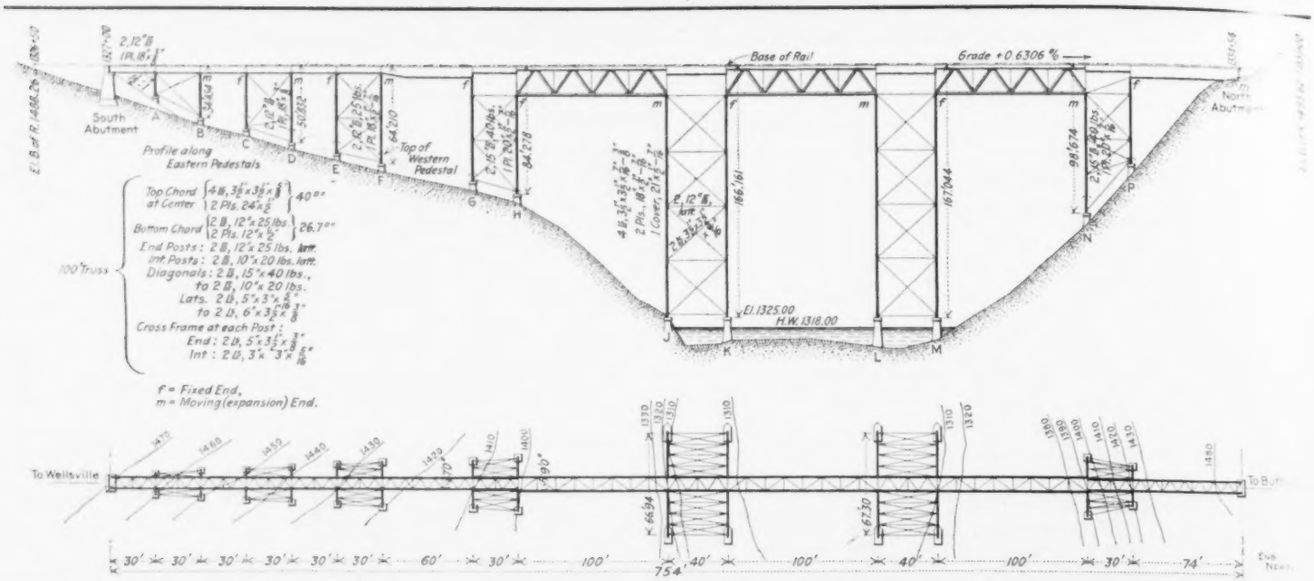

FIG. 1. GENERAL ELEVATION AND PLAN OF BUFFALO & SUSQUEHANNA RY. CO. VIADUCT OVER CANEADEA GORGE, NEAR CANEADEA, N. Y.

The tower NP, which carries a 74-ft. and a 100-ft. span of different depths, offered some difficulties in carrying out the avoidance of skew connections. In order to make the tower wedge-shaped, that is to say, to make the longitudinal bracing lie in planes parallel with the axis of the viaduct, both bents were capped at the same level, a level fixed by the base-plate of the deeper (100-ft.) span. The 74-ft. span is supported on a short superimposed bent (see Fig. 4), whose posts have a steeper inclination than the standard 1 to 5 batter of the lower bent. Difficult splicing is avoided by fitting the bottom of the upper bent with a regular footing, which rests on a horizontal cap of the lower bent. The superimposed bent is braced longitudinally by a diagonal strut NP. As this strut connects at one end to the posts of bent N, which have a 1 to 5 inclination laterally, and at the other to the posts of the upper bent at P, which have an inclination of 1 ft. in 11 ft. 4¼ ins., direct connections would be badly skewed. To simplify them, the strut is fitted with cross beams at N and P, Fig. 3, and these beams bear angle lugs which engage gusset-plates on the posts, as shown.

The longitudinal and cross bracing of the towers consists of latticed struts and of tension members made of two angles connected by tie plates. Wherever practicable, and this is almost everywhere the case, the center lines of all the members of the towers and 100-ft. spans meeting in a junction exactly intersect in one point so as to avoid secondary bending stresses due to eccentric connections. In many designs such secondary stresses amount to a large fraction of the primary stresses, although the latter are generally the only ones considered in the calculation.

The steelwork of this viaduct (and in fact of all the bridges for this line of railroad) was designed and the inspection superintended by the writer. The American Bridge Co. constructed and erected it.

Mr. E. P. Lupfer, M. Am. Soc. C. E., Assistant Chief Engineer, Buffalo, N. Y., had charge of the construction of the entire line from Wellsville to Buffalo and all concrete substructures were designed in his office.

The concrete work on this viaduct, in fact of the whole line was built by the Continental Engineering and Contracting Co., of Buffalo, with Mr. Clarence R. Neher, M. Am. Soc. C. E., in charge.

Mr. H. Herden, of Galeton, Pa., is Chief Engineer of the Buffalo & Susquehanna Ry.

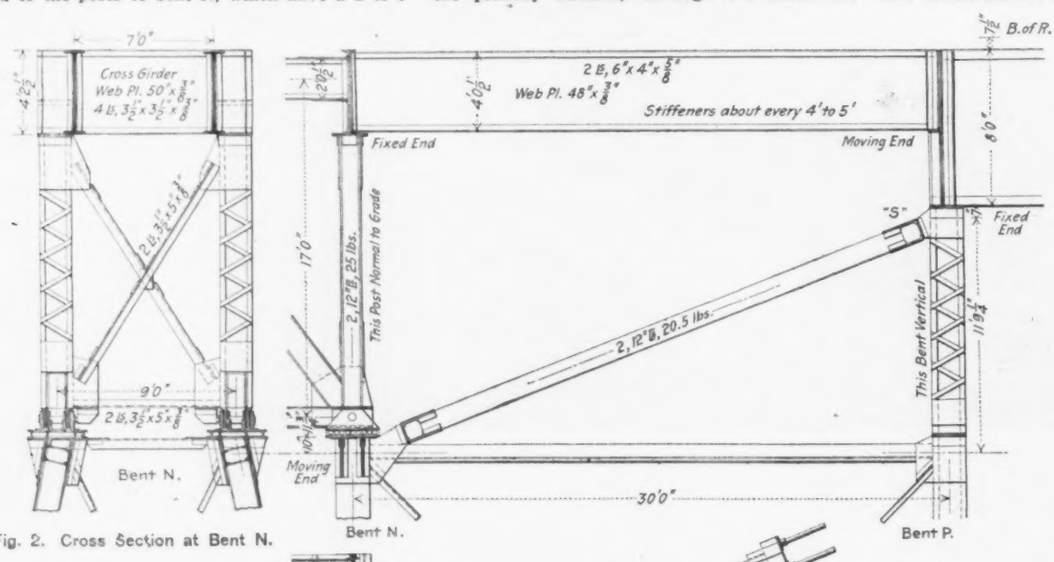

Fig. 2. Cross Section at Bent N.

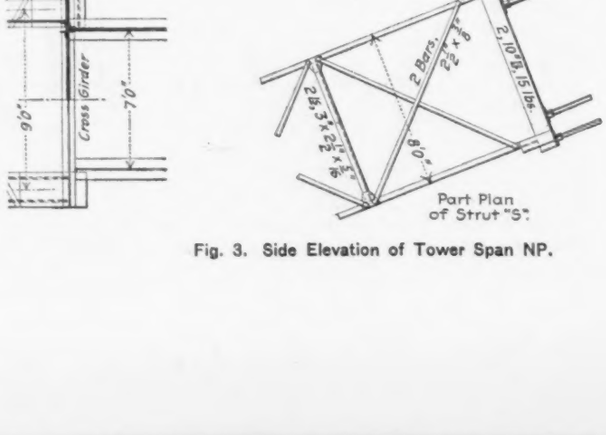

Fig. 3. Side Elevation of Tower Span NP.

FIGS. 2 TO 4. DETAILS OF CANEADEA VIADUCT.

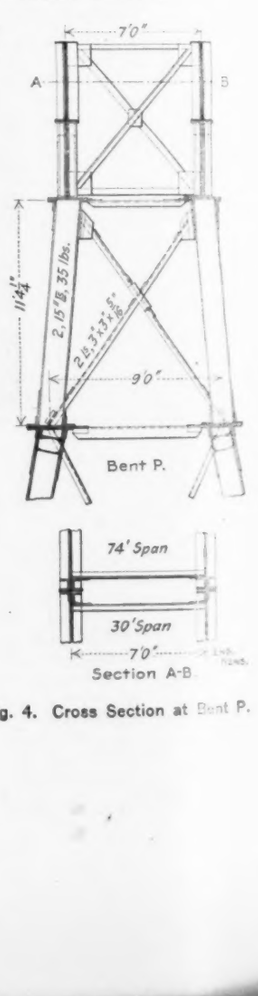

Fig. 4. Cross Section at Bent P.

FIG. 5. VIEW OF CANEADEA VIADUCT DURING ERECTION.

THE TIMBER CREOSOTING PLANT AT SHIRLEY, IND.*

By H. H. Knowlton.†

The [creosoting] plant at Shirley, Ind., is owned by the Columbia Creosoting Co., of which L. T. Dickerson is President, and C. B. Lowry is Manager. The Rueping process is employed, which consists in the injection of creosote oil under pressure into seasoned timber to a certain amount per cubic foot of timber, and the removal by vacuum of surplus oil.

Before these ties are treated they must be thoroughly seasoned, as ties with sap or water in them are not penetrated to any depth by the oil. Many plants are built to thoroughly steam the timber and to pump the sap out of it, this process taking place in the same cylinders as the injection of the oil and immediately preceding it. Others resort to a kiln to drive all moisture of any character away. This plant has no provision for either of these. The particular point is made by the company that steaming timber reduces its strength by from 10% to 40%; that it encourages splitting and warping; that it does not remove all sap and water, and for these reasons its process is much superior.

The general plan of the plant is a seasoning yard at the north end of the retort house, with two railway tracks 93 ft. c. to c., running its full length, about 1,500 ft. South of the retort house four electric tracks run between the other two tracks, which are extended past the retort house and on each side of a tie dock or platform. The two electric tracks on the outside are for placing buggies to be loaded and unloaded, and the two inside tracks are for the purpose of storage of loads and shifting. South of the retort house is the tie dock, with timber sides and filled with earth to the height of a car floor; and there is room for ten railway cars on each side of it. Seasoned ties when shipped in are unloaded directly from the cars onto the buggies, and treated ties are unloaded directly from buggies onto railway cars. Five cars inbound and five cars outbound are handled at one time on each side, and as buggies are unloaded of treated ties they are run up to cars with inbound ties on them, and are loaded for treatment. The buggies have roller bearings and are easily shifted by hand even when loaded.

The yard for the storage and seasoning of ties has about seven acres of available ground, which allows room for the storage of 500,000 ties, space being reserved for loading and unloading tracks and for men to handle ties. The ties are piled in a systematic manner, and to best advantage to season thoroughly. Also the ties of same kind of wood and age after cutting are piled together and so that all ties treated in one run are as nearly as possible seasoned to the same degree and are of the same kind of wood, or of woods of about same density. In order to assist in seasoning, a standard method of piling is adopted, which is to lay two creosoted ties on the ground, on top of which eight green ties are piled with uniform spacing between them, the outside two being placed on edge and the others flat. In the next tier are placed two ties only, and then eight as before, continuing until the pile is 12 to 16 ft. high. Another method of piling, with greater economy of room, is to place a tie at one end only on alternate tiers, which makes the solid rows sloping. By both methods air can circulate freely and ties can thoroughly season in the least possible time. Each pile of ties is marked to show age, when cut, where from, and whether grown on high or low ground, all of which information is of utmost importance to secure proper treatment.

From eight months to one year is required to thoroughly season ties (with dense oaks two years would be better) and render them fit for treatment, and here appears the most serious objection to this process, and that is the tying up of a large amount of money for a year. In case the number of ties for treatment was 350,000 the railway would have not less than $100,000 invested in ties all assembled at one place and menaced by fire.

When properly seasoned the ties are loaded on buggies made entirely of metal and run directly into the retort. These buggies operate on a track of 30-in. gage, with 52-lb. rails, and are shifted in trains by means of an electric motor car, which weighs about six tons and which depends upon a conductor rack laid between the rails for its tractive power. The maximum drawbar pull is 850 lbs. It has two speeds, 85 ft. per minute and 350 ft. per minute, change being made in the gears. This rack is about 2 ins. above top of rails, and is secured to the ties by spikes, insulation being interposed. It is protected along the sides by wood strips, which extend ¾-in. above the rack. This seems to provide ample insulation against accidents of any nature. At switches the rack broadens to triangular form, to which the movable section of the rack is attached for each of the different routes as required. Bonding through movable sections is secured by the usual means.

There are 15 buggies handled in each train, this being the exact number contained by the retorts. These buggies are 4 ft. long and ties are piled longitudinally on them, care being taken to have them open piled to insure thorough treatment. Two curved ribs retain the pile, and when completely loaded the movable part of rib is placed in position, completing the circle and guarding against too big a load, which could not be run into the retort. Four lugs are riveted on the under body of the car, which engage an angle iron inside the retort and prevent derailments. They have no couplers and when pulled have a cable run through a loop on each car, so that the whole load is pushed by the car farthest from the motor. The total number of buggies in use is 200, which allows any combination in way of loading which may be desired. With the plant at full capacity 30 laborers are employed handling the treated ties from the buggies and loading them on cars, and loading the untreated ties from the piles in the yards or from cars. These men are paid ¾ ct. for each tie handled; they work in gangs of four. In order that the handling of ties may be done economically, the plant is not run unless cars are provided upon which to load the treated product, no provision being made to store treated ties. As treated ties burn fiercely, it can readily be appreciated why it is undesirable to have them upon the premises in any quantity.

The injecting of oil is done in two large retorts placed side by side. The building housing them is fireproof, with cement floor and slate roof, supported by steel trusses. The retorts are cylindrical, 7 ft. diameter, 130 ft. long, with ⅝-in. shell; each will contain 15 buggies at one time, each buggy having 52 to 55 ties, which gives capacity for one loading of about 800 ties.

Through these retorts run the tracks, and the trains are run directly in at one end of the cylinder. This entire end of the retort is a cast steel door, which fits to a flange on the end of the retort, being swung on two large hinges and being supported at the bottom by a wheel. When closed it is secured by 50 bolts 2 ins. diameter, which are hinged on one end, so that they may be swung free after the nuts are loosened. To open the head, the nuts are started with a friction wrench and then run back by hand, but not taken entirely off, the object being to save time. Two men can open the door and swing it in eight minutes. As soon as the door is open two pieces of rail 10 ft. in length are placed in position from the track inside the cylinder to the track in the yard, and the cylinder is in condition to load or unload.

Through the retorts and underneath the track are run eight lengths of 1½-in. double strength pipe, through which live steam is passed to maintain the temperature of the oil at from 165° to 180° F., as a necessary part of the process is the heating of the oil during the time of injection. In order to allow for expansion and contraction of the retort due to variation in temperature, sliding plates and saddles are provided. The two center saddles upon which it rests are bolted to the concrete foundation, and all others are free to slide on metal plates imbedded in concrete pedestals, the saddles being riveted to the shell.

The creosote oil is obtained principally from domestic gas plants, tar works and similar plants, and is shipped to Shirley in tank cars of about 8,000 gals. capacity. A special track is provided for placing these cars, and the oil is pumped directly from the cars into two large receiving tanks through a 5-in. main, through which is run a live steam pipe. These two receiving tanks are cylindrical, and built directly upon heavy concrete foundations. They have each a capacity of 180,000 gals. Each has 56 coils of 1½-in. steam pipe about 12 ins. from the inside of the shell to keep the oil liquid, so that it may be freely pumped and not clog the mains.

The distance between the pump house and tanks is 525 ft., and the pipe line is 5 ins. diameter. This combination causes some trouble, as the oil is very heavy (specific gravity 1.02½ to 1.07), and flow is greatly retarded. Directly over the retorts and extending above the roof are two tanks of 30,000 gals. capacity each, and the oil is supplied to them from the main storage tanks. The oil mains are 10 ins. diameter, and duplicate valves are provided to guard against a shutdown in case one breaks. Through these overhead tanks the exhaust steam is run and the temperature of the oil is maintained at 135° to 180° F., by means of numerous live steam coils in addition to the exhaust steam. There is also just outside the retort house an underground steel tank 100 ft. long and 8 ft. diameter, into which the oil from the retorts is returned at the completion of a run. This tank is necessary to catch the dirt and debris from the ties and to save time in emptying the retorts.

The pump which supplies oil to the overhead tanks is so connected that it returns the oil from the underground tank to the overhead tanks; it will pump from the main storage tanks to the overhead, or from cars to overhead tanks or to main tanks. This is another safeguard against a breakdown. The vacuum pump for the removal of a surplus oil from ties is connected to the top of the retort by a main which passes through a condenser. The gages show that with oil at 165° F. it is possible to get a 20-in. vacuum in 20 minutes and 25-in. vacuum in 45 minutes, all of which is due to the perfection of the condenser, which I am not at liberty to describe. This is a remarkable performance. The boiler room is located midway alongside the retort house and about 30 feet from it. The boilers are of the return tubular type and are rated at 200 HP. Natural gas is used as fuel.

The following force is required to work one shift: One man at the boilers, one head man in the retort house with three men as his assistants, 12 to 16 laborers for day work only. With a double shift it would require 30 laborers, who work during daylight only. A machinist and general all-around man is also on duty.

The process is as follows: The retort is loaded with seasoned ties and the movable end secured. Oil at a temperature of 135° to 180° F. is then turned in from the auxiliary tanks above the retorts until all voids are filled, which requires about 20,800 gals. By means of a circular gage, which has two hands showing the amount of oil in the overhead tank to a reading of 2 gals., and by taking a reading before turning the oil in and one when the retort is full, the exact amount of oil turned into the retort is measured. A record is then made of the number of run, time, number of ties, kind of wood and amount of oil required to fill interstices.

The gravity supply from the auxiliary tank is then cut off, the pressure pump cut in, and oil pumped from the

*Abstract of a paper in the annual proceedings of the Indiana Engineering Society for 1906.

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same overhead tank until the pressure reaches 180 lbs. is maintained for 3½ to 6 hours. A Bristol automatic recording gage makes a permanent record for each 24 hours of the pressure, and the hours and minutes maintained. The card is removed at the end of the day, properly numbered and filed. The total amount of oil put into the retort is measured by the gage above described. After the time required for treatment has elapsed, which is a matter of judgment based upon the amount of oil forced in, the kind of timber, and the rate of absorption, the pressure is cut off and relieved by a small by-pass valve discharging into the 10-in. main leading to the underground tank.

At atmospheric pressure the 10-in. valve is opened and the surplus oil in the retort is rapidly discharged into the same underground tank. All valves are then closed and the vacuum pump is started, and a vacuum of 25 ins. is maintained until it is determined that a proper amount of oil has been released from the ties. Without this vacuum process, the ties would exude oil for many hours after the pressure was released, as the expansion of air within the ties would force out a considerable quantity of oil, which would be lost. On a hot day this loss is especially noticeable. Also, by the very high pressure maintained the oil is forced to the very heart of most of the woods treated and a thorough impregnation assured, the time of treatment is shortened, and still an excess of oil is not left in the tie. The surplus oil brought out by the vacuum from ties drops to the bottom of the tank and is drained to the underground tank.

After the completion of the process, no determination of the exact amount released is possible until the supply pump has returned all oil from the underground tank to the overhead tank, in which it is again measured. This is done as quickly as possible, and the amount of oil taken up by the ties is then figured. As the load has been removed, the proper time to cease treating is wholly a matter of judgment based upon experience and assisted by the records of the pressure pump and vacuum pumps, as shown by the automatic recording gage. Should the amount contracted for per tie not appear from the figures, the load must be returned for further treatment.

To get the average amount per tie, the total amount taken up is divided by number of ties in the run, all of which is on record. To get the amount of oil per cubic foot of timber the company uses a method of approximation. A large number of ties were measured carefully and the average cubic contents for the different sizes of ties ascertained. These figures are used in all runs to get the amount of oil injected per cubic foot of timber. As the company guarantees a certain quantity of oil per tie, the process is an inverse one and the calculation is made beforehand of the amount of oil required to properly treat the load and the surplus is removed by means of the vacuum pump.

One sample tie is removed for each run made and identified by a metal tag. If an inspection of results secured is desired a number of ties are taken after treatment, sawed in two, split in sections and observations made upon the penetration of oil, which is easily detected by discoloration of the wood, as it leaves a dark stain. It might be suggested that the amount of oil in a tie could be determined by weight, but this is not as simple as it appears. Before treatment a tie may be weighed, but we cannot determine the amount of moisture in it which will be driven out by immersing it in hot oil for six hours at a temperature of 165° F. under pressure of 180 lbs. Further than this, the distribution in the tie would be a matter of conjecture only, and the sap wood might have it all. A cross section of tie shows the distribution perfectly. The railway company has a man always at the plant who checks the reading of the gages. As individual ties of the same wood, apparently alike in their essential features, vary in absorptive power, it follows that one tie gets much more than another, but the average amount is there, and probably more than 80% of the ties will show substantially the same results. The railway company's man has also to keep a record of cars of ties received, the inspection of them, and the loading and billing out of cars. The creosoting company stands the expense of loading and unloading cars, and the railway company of the switching and hauling. There is no variation from this in the storage yard.

As all records of each treatment are made from the gages they deserve especial mention. Each retort has a pressure gage, a vacuum gage and a temperature gage; these are automatic recording gages. Reference may be made to them for any run at any time. As a check on them, there is also an ordinary pressure gage and vacuum gage placed directly below them. These ten gages, five for each retort, are assembled on one board together with a steam pressure gage. Directly alongside this board are located the pumps and all valves for the proper distribution of oil. Above and in plain view are the gages which measure the oil from the overhead tanks. Each overhead tank and main tank is provided with a thermometer which projects 4 to 6 ft. into the interior, as the temperature of the oil is a most important matter. Each retort is provided with a thermometer in addition to the temperature gage above described, which serves as a check on its readings.

Wet or unseasoned timber cannot be successfully treated, as the wood cells are already occupied and cannot take up the oil. In detail the treatments of various woods appear as follows:

Beech	sapwood perfectly, heart of young good, and of red heart old, not at all.
Birch	good.
Hard maple	perfectly, but hard to treat.
Soft maple	perfectly, but it is of no service.
Red oak family	perfectly.
Sycamore	perfectly, but unsuited for ties on account of its brittleness.
Black gum	perfectly, is very tough and never splits, but tie plate must be used, as it is soft.
Sweet gum	sap perfectly, red heart difficult, is not as tough or strong as black gum.
Red elm	perfectly, is tough and soft.
White elm	perfectly, is soft and not strong enough for tie.
Rock elm	perfectly, is hard and strong.
Hickory	perfectly and makes good ties.
Ash	perfectly and makes good ties.
Loblolly pine	perfectly.
Short leaf pine	sap perfectly, heart not good.
Black pine	perfectly.
Long leaf yellow pine	cannot penetrate heart.
Tamarack	sap perfectly, but cannot penetrate heart.

The only woods now being treated are the red oak family, beech, hard maple, hickory, ash and rock elm.

Some casual observations made at various times show that bark forms a perfect protection against treatment. Many ties are thoroughly impregnated and apparently have an overdose. Some ties are penetrated only to a limited extent, but as the destroying bacteria and fungus must all come from the outside, the treatment may be sufficient. No observation may yet be made of the lasting qualities of ties turned out by the company, as they have been running only about a year, and substantial conclusions cannot be drawn until ties have been in actual service at least 16 years. The cost of treatment is a matter of much interest. The rates are about 30 cts. per tie for 2½ gals. of oil per tie. The oil usually costs about 6 cts., although special contracts with guaranteed amounts may reduce this figure, but this is something which we cannot arrive at accurately. The plant represents an outlay of about $75,000, and the cost of operation may be approximated from the figures given.

There is usually a large perentage of loss in creosoting plants from the evaporation of oil. In this plant the oil is confined at all times, and consequently there is practically no evaporation, even the gases removed by the vacuum pump being condensed and the oil recovered. This is possible only by reason of the specifications under which oil is purchased, which is, that it must be a pure product of coal tar distillation, free from adulterations of other oil or substances, and when tested must have an initial boiling point of 220° C., and must distill off approximately 80% at 315° C., and the residuum above 355° C. must be soft and plastic. A variation of not more than 5% will be allowed from above distillation. The company puts little faith in analysis and none in specific gravity alone, as pitch might be added to increase it. The oils accepted run from 1.02 to 1.07½, and average near 1.05½. A chemist is constantly employed in analysis and experiments.

As to cost of ties, the woods accepted untreated run from 65 to 75 cts. per tie, and their average life is about nine years. This is a very high price, and ten years ago the Cleveland, Cincinnati, Chicago & St. Louis Ry. was purchasing ties better than it gets to-day, which cost but 40 cts. apiece. The specification under which ties for treatment are now purchased is a very rigid one, and the cost of these ties untreated is 35 cts. Add to this the cost of treatment, say 30 cts., and you have a tie which costs 65 cts., exclusive of cost of loading, transportation, switching and capital invested, which considerably increase it. In order that this treated tie should last its mechanical life and not be subject to any rot, it will be important for railways to secure as nearly perfect drainage for their tracks as possible, so that the oil may not be washed out and that the tie may not be rendered soft and be cut into by the rail. The present practice of American roads which use the ordinary driven spike is most destructive to all ties, as the fiber of the wood is crushed and a hole made to hold moisture and assist in rotting out and softening the tie. The withdrawal of the spike and resetting it in gaging the track actually weakens a tie to such an extent that many ends of sound ties break off, necessitating their removal. Another practice which is most pernicious is the use of our present form tie plates, especially those with claws or ribs which cut into timber. It is my opinion that we cannot realize the proper benefit from preservative treatment of ties until we substitute a screw spike and a smooth tie plate, which I look upon as essential adjuncts in the use of treated ties. By this means there is no reason why we cannot raise the average life of ties from nine years to 16 years.

SOME TABLES AND OTHER DATA FOR RAILWAY LOCATING ENGINEERS.

By C. P. Howard,* M. Am. Soc. C. E.

(With inset sheet.)

In projecting location, and in making approximate estimates and comparisons, it is important to have a set of tables in convenient form so that calculations of cost can be made in the shortest possible time and with a reasonable degree of accuracy.

Perhaps the most convenient work sheet form for excavation and embankment tables is to copy them on slips of profile paper, so that quantities can be scaled from the profile. In the mountains it is necessary to use tables calculated for different slopes of ground, and, although it is easy enough to make them, that is, to know how, they are tedious to calculate, especially if the engineer happens to be in camp and rushed for time. Therefore Tables I., II., III., for different slopes are given which, copied on slips of profile paper, can be used to scale quantities from the profile. In very rough country, where a large proportion of solid rock is expected, ½ to 1 excavation tables may be used to advantage. There is not much error in using level cuttings for ½ to 1 side slopes, even on steep ground; but for fills and flatter cut slopes, tables or diagrams (see Table IV.) for different ground slopes are necessary.

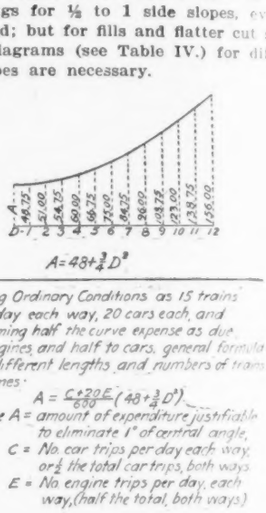
A=
D-1 2 3 4 5 6 7 8 9 10 11 12
A=48+¾D²
Taking Ordinary Conditions as 15 trains per day each way, 20 cars each, and assuming half the curve expense as due to engines and half to cars, general formula for different lengths and numbers of trains becomes:
A = C+20E/6.88 (48+¾D²)
Where A = amount of expenditure justifiable to eliminate 1° of central angle,
C = No. car trips per day each way, or ½ the total car trips, both ways
E = No. engine trips per day, each way, (half the total, both ways.)

Fig. 1. Diagram and Formula Showing Amount of Expenditure Justifiable Under Ordinary Conditions, to Eliminate 1% of Curvature.

Table V, for cubic yards in abutments, is based on a wall 2 ft. wide at top, 3 in 12 batter on back and ½ in 12 batter on front. The base, for heights given in table, will be somewhat over one-third of the height. The writer has never known an abutment with proper foundations and these dimensions to fail. Of course the toe of the abutment, if not on solid rock, should be made absolutely safe against settlement by spreading the bottom courses or by some method of artificial foundations.

Table VI. giving cubic yards of masonry for box culverts is taken from one used by Chesapeake & Ohio Ry. locating parties.

For arch culverts see Trautwine, page 626, 1902.

Table VII. A, giving weight of steel in single track bridge spans, seems to have been based on Cooper's E 40 loading, and was furnished by the American Bridge Company about four years ago. The rule for approximate weight of viaducts is particularly handy. More recent information, on the subject, from the same company (1906) is given in Table VII. B. Table VII. C, giving weight of steel for the single and double-track spans, has been calculated for spans up to 550 ft. from formulas (which follow) Nos. 1, 2, and 4, of Mr. E. K. Morse, C. E., Pittsburg, Pa., and is based on loading similar to Cooper's E 50, but with particularly heavy loads on cars, suit-

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able for handling metal around Pittsburg. These formulas have been worked backwards from actual construction, and if there is any criticism to be passed on them, it is that they are considered too close an approximation, and not as liberal as might be desired. The "Hot Metal Bridge" applies to a structure with a solid floor system and sides sufficient to provide protection for river and other travel underneath same during the transit of hot metal from furnace to mill.

Mr. Morse's formulas are as follows:

(1) Ordinary pin connected, through truss, open floor system, single track, $W = 7\frac{1}{2} L^2 + 500 L.$
(2) Do. Double track, $W = 12 L^2 + 1{,}000 L.$
(3) Double track, through truss, pin connected, solid floor system of blast furnace sand; hot metal bridge, $W = 14 L^2 + 2{,}800 L.$
(4) Ordinary deck plate girders, single track, open floor, $W = 10 L^2 + 350 L.$
(5) Do. Hot metal, $W = 12 L^2 + 1{,}500 L.$
Where W = total weight and L = extreme length.

Another formula has been suggested for single track spans of 100 ft. to 400 ft., using Cooper's E 50 loading and the American Bridge Company's Specifications; and is based on economical depths of stringers, trusses and floor beams:

$$W = (585 + 7.45\ L - 0.001\ L^2)\ L.$$

It gives weights that differ little from Table VII. C, viz., 100 ft. span, 132,000; 200 ft., 407,000; 400 ft., 1,362,000. Another formula suggested for single-track spans up to 350 ft., using Cooper's E 50 loading, is, $W = (8.5\ L + 550)\ L.$ This gives somewhat greater weights than the others.

TRESTLES.

The timber in trestles may vary greatly according to design. For an approximate estimate of the feet B. M. in framed trestles, 25 to 90 ft. high, 12½ ft. spans, c. to c. (having three 9 × 16-in. stringers under each rail, four 12 × 12-in. posts in the upper deck, six 12 × 12-in. posts in each lower story, and suitable bracing), multiply the square feet of profile area by 13. Multiply the feet B. M. by $^7/_{100}$ to get the pounds of iron. For a trestle 20 ft. high multiply by 15 instead of 13. That is, a trestle 50 ft. high would have $13 \times 50 = 650$ ft. B. M. per running foot of trestle, and $^7/_{100} \times 650 = 45$ lbs. of iron per running foot. If a trestle is designed with four posts in the upper deck, and a forest of posts in the lower decks, no multiplier based on the square feet of profile area can be expected to hold good. It may be well to bear in mind in designing trestles that the lower decks have no more weight to carry than the upper, except for the additional weight of timber.

ELIMINATION OF CURVATURE.

The diagram and formula (Fig. 1) for amount of expenditure justifiable to eliminate 1° of curve was deduced somewhat arbitrarily, and is designed mainly for freight business. For a road intended to handle a fast passenger service, it might be advisable to allow somewhat higher values. The curve of values varies as (¾ of) the square of the degree of curve, except that it starts at $48 instead of zero. It allows about three times as much for eliminating curvature on a 12° as it does for a 2° curve. The formula for different lengths and numbers of trains is $A = \frac{C + 20\ E}{600}\ (48 + \frac{3}{4}\ D^2)$, which in plain English means that for trains of 20 cars, half the curve expense is due to engines and half to cars; and that doubling the number of cars will add 50% to curve expense, and doubling the number of engines will do the same. Taking the ordinary or average conditions as 15 trains per day each way, 20 cars per train, and using the values given in the diagram, we have $75 for a 6° curve, and $96 for an 8° curve. For these conditions we get from Wellington, page 323, par. 357, $81.15, though he uses 8% interest to get it; and from Mr. J. B. Berry's formula (Union Pacific R. R.), $0.19½ × 30 ÷ 5% = $116. Both of these values are independent of the radius. For 3° and 4° curves our formula would be considerably less, and for 11° and 12° curves considerably higher.

SPIRAL CURVES.

The following is an abbreviation of a method by Mr. J. R. Stephens, of Germantown, Philadelphia, which is designed to simplify the use of spirals.

As suggested by Mr. Shunk, the main curve is compounded with a curve of twice the radius. This curve of twice the radius, or "Terminal Curve," is made of such a length that it gives the same tangent, nearly coincides with, and can be replaced by, a spiral. Consequently for the ordinary purposes of location and grading, the terminal curve is all that is necessary.

When track centers are run, corrections for the exact spiral can be made by small offsets. This may be done as follows: Fasten a small wooden scale to an octagonal transit rod by means of a cord or rubber band. The scale should be about 0.6 ft. long, marked in tenths and twentieths, and fastened across the rod near its foot like the arm of a cross, so that the center of the scale coincides with the center of the rod. Then the small offsets can be read by the transitman from his instrument, the same as with a self-reading level rod. These offsets, however, need not be made except for track centers, as the compound curve closely approximates the spiral. Probably none of the offsets for an exact spiral will exceed 0.2 ft. For spirals 210 ft. long the maximum offset from the terminal curve to spiral would be 0.09 ft. for a 4° curve, and 0.18 ft. for 8°; for spirals 240 ft. long, 0.12 ft. for 4°, and 0.25 ft. for an 8° curve.

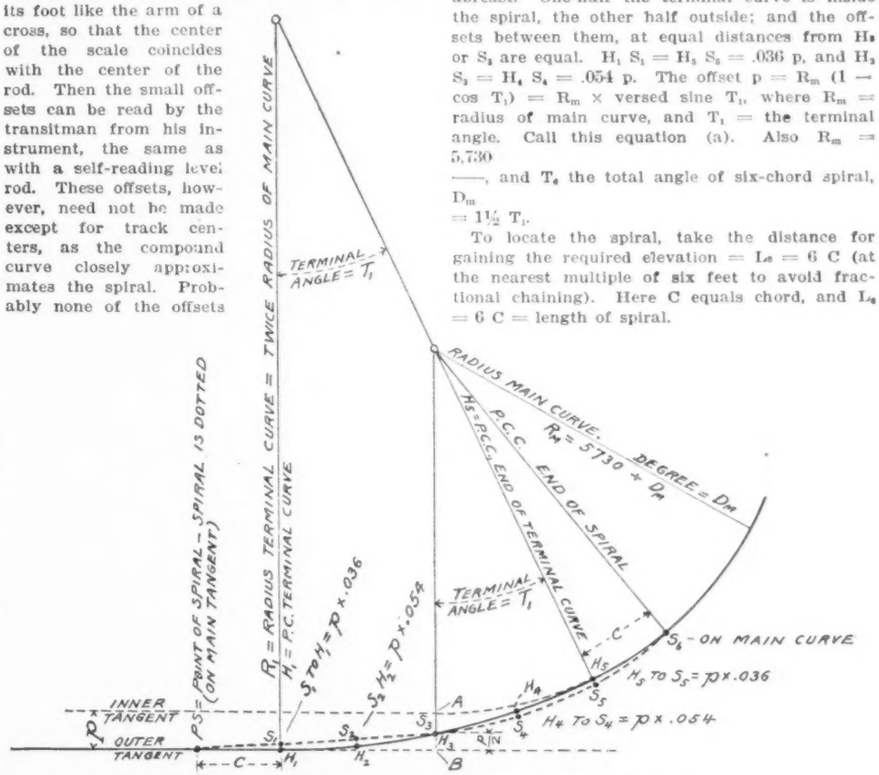
R = RADIUS TERMINAL CURVE = TWICE RADIUS OF MAIN CURVE
H = P.C. TERMINAL CURVE
P S = POINT OF SPIRAL — SPIRAL IS DOTTED (ON MAIN TANGENT)
TERMINAL ANGLE = T1
RADIUS MAIN CURVE = 5730 ÷ D
DEGREE = D
P.C.C. END OF SPIRAL
P.C.C. END OF TERMINAL CURVE
S1 TO H1 = P × .036
S2 H2 = P × .054
TERMINAL ANGLE = T1
S6 - ON MAIN CURVE
H5 TO S5 = P × .036
H4 TO S4 = P × .054
INNER TANGENT
OUTER TANGENT
C
A
B

FIG. 2. STEPHENS SIX-CHORD SPIRAL AND TERMINAL CURVE.

The terminal curve is always exactly ⅔ the length of the spiral, so that if you want to put in a spiral 240 ft. long on a 6° curve, you can compound with a 3° curve 160 ft. long, substituting the spiral when you put in track centers. If preferred, the exact six-chord spiral can be run in by deflections, first putting in points, P S and S_6 on main tangent and main curve. (See Table IX.).

THE STEPHENS SIX-CHORD SPIRAL AND TERMINAL CURVE HAVING A RADIUS TWICE THAT OF THE MAIN CURVE.

FORMULAS:

(1) $p = R_m\ (1 - \cos T_1)$ = main offset between tangents.

(2) $R_m = \frac{p}{1 - \cos T_1}$ = radius of main curve.

(3) $\cos T_1 = \frac{R_m - p}{R_m}$; where T_1 = terminal angle.

(4) $p = .436\ D_1 \left(\frac{L_1}{100}\right)^2$

(5) $L_1 = 151.5 \sqrt{\frac{p}{D_1}} = \frac{2}{3} L_6$ = length of terminal curve.

(6) $L_1 = 2\sqrt{pR_1} = \frac{2}{3} L_6$

(7) $p = \frac{L_1^2}{4 R_1}$

(m) written below refers to the main curve.

(1) written below refers to the terminal curve, or "one-chord spiral."

(6) written below refers to the six-chord spiral.

This spiral (see Fig. 2) has six chords, each = C = one-fourth length of terminal curve; hence the spiral is 1½ times the length of terminal curve, and the quarter points H_1 H_2 H_3 H_4 of the terminal curve, are abreast the one-sixth points, S_1 S_2 S_3 S_4 of the spiral, while S_3 and H_3 coincide. $S_3A = S_3B = \frac{p}{2}$, and H_4 and S_5 are abreast. One-half the terminal curve is inside the spiral, the other half outside; and the offsets between them, at equal distances from H_3 or S_3 are equal. $H_1 S_1 = H_5 S_5 = .036$ p, and $H_2 S_2 = H_4 S_4 = .054$ p. The offset $p = R_m\ (1 - \cos T_1) = R_m \times$ versed sine T_1, where R_m = radius of main curve, and T_1 = the terminal angle. Call this equation (a). Also $R_m = \frac{5{,}730}{D_m}$, and T_6 the total angle of six-chord spiral, $= 1\frac{1}{2}\ T_1$.

To locate the spiral, take the distance for gaining the required elevation = $L_6 = 6$ C (at the nearest multiple of six feet to avoid fractional chaining). Here C equals chord, and $L_6 = 6$ C = length of spiral.

Then $\frac{2\ C \times D_m}{100} = T_1$

where D_m = Degree of main curve, T_1 = Terminal angle in degrees, C = Length of chord in feet.

Next calculate p from equation (a); run in the terminal curve, and offset to spiral. Locate the point of spiral (P, S) and S_6 on outer tangent and main curve, one chord length from H_1 and H_5 respectively.

Example.—Take a 14° curve having a spiral approach of six chords, each 25 ft. long or 150 ft. in all, in place of which a 7° terminal curve, 100 ft. long has been used; to calculate the offsets to spiral.

$R_m = 5{,}730 \div 14 = 409.3$; $L_6 = 25 \times 6 = 150$.

T_1 (the terminal angle) $= \frac{2C \times D_m}{100} = 50 \times 14 \div 100 = 7°$.

The main offset $p = R_m\ (1 - \cos T_1) = 409.3 \times .00745 = 3.05$ ft.

The offsets $H_1S_1 = H_5S_5 = 3.05 \times .036 = 0.11$ ft.
$H_2S_2 = H_4S_4 = 3.05 \times .054 = 0.16$ ft.
$H_3S_3 = 0.0$.

The P S and S_6 are set as shown in Fig. 2.

The 7° approach from H_1 to H_5, or terminal curve, will be four 25-ft. chords. Whenever in-

TABLE VIII.—INTERMEDIATE OFFSETS TO THE SIX-CHORD SPIRAL FROM MAIN TANGENT, MAIN CURVE AND TERMINAL CURVE.—To be measured inward for the main tangent half of spiral and outward for the main curve half.

Tenths of chord length.	Coefficients which multiplied by p give offsets in feet.	Tenths of chord length.	Differences for each hundredth of chord length.	Tenths of chord length.	Coefficients which multiplied by p give offsets in feet.	Tenths of chord length.	Differences for each hundredth of chord length.	Tenths of chord length.	Coefficients which multiplied by p give offsets in feet.	Tenths of chord length.	Differences for each hundredth of chord length.
P S	.000	S_6	.0000	S_1	.036	S_5	.0006	S_2	.054	S_4	.0004
1	.000	9	.0001	1	.042	9	.0006	1	.050	9	.0004
2	.001	8	.0002	2	.048	8	.0004	2	.046	8	.0005
3	.003	7	.0003	3	.052	7	.0004	3	.041	7	.0005
4	.006	6	.0003	4	.056	6	.0002	4	.036	6	.0005
5	.009	5	.0004	5	.058	5	.0001	5	.031	5	.0005
6	.013	4	.0005	6	.059	4	.0000	6	.026	4	.0006
7	.018	3	.0005	7	.059	3	.0000	7	.020	3	.0006
8	.023	2	.0006	8	.059	2	.0002	8	.014	2	.0007
9	.029	1	.0007	9	.057	1	.0003	9	.007	1	.0007
S_1	.036	S_5		S_4	.054	S_4		S_3	.000	S_3	

TABLE IX.—DEFLECTION COEFFICIENTS FOR SIX-CHO[illegible] SPIRAL.—These coefficients multiplied by (C × D), where C equals chord leng[illegible] feet, and D equals degree of main curve in degre[illegible], give deflections of spira[illegible] m tangent at transit in minutes and decimals.

Coefficients for each position of transit are along the horizontal l[illegible]

Transit over..	P S	S_1	S_2	S_3	S_4		S_5
P.S	.....	.0429	.1071	.2000	.3214	.4[illegible]	.4500
S_1	.0429	.....	.0857	.1929	.3286	.4[illegible]	.4857
S_2	.1500	.0857	.....	.1286	.2786	.4[illegible]	.5143
S_3	.3143	.2357	.1286	.....	.1714	.3[illegible]	.5857
S_4	.5357	.4429	.3214	.1714	.....	.2[illegible]	.4500
S_5	.8143	.7071	.5714	.4071	.2143	..	.2571
S_6	1.1500	1.0286	.8786	.7000	.4929	.25[illegible]	.....

Total angle in minutes P S to $S_6 = C \times D \times 1.8$. The degrees [illegible] urvature of the spiral arcs are successively $\frac{D}{7}$; $\frac{2D}{7}$, etc., to $\frac{6D}{7}$. The [illegible]ath are would coincide with the main curve.

termediate offsets are required, as in centering trestle bents, etc., Table VIII. is used.

Example.—In the preceding example let the point of spiral (P S) be at station 7 + 07, chords 25 ft., required the offset at the even station 8. The curve may be tabulated thus:

P S = 7 + 07
S_1 = 7 + 32
S_2 = 7 + 57
S_3 = 7 + 82
S_4 = 8 + 07
S_5 = 8 + 32
S_6 = 8 + 57

Hence 8 = S_5 + 18/25 = S_5 + 0.72 toward S_4, which by interpolation in Table VIII = .042 and, .042 × p or 3.05 = .128 ft.

If the numbering ran in the opposite direction, the offset at 6 + 40 being required then,

P S = 7 + 07
S_1 = 6 + 82
S_2 = 6 + 57
S_3 = 6 + 32

Here 6 + 40 = S_3 + 8/25 = S_3 + 0.32 toward S_2 which by Table VIII. = .0212 × 3.05 = .074 ft.

The following rule is suggested for determining the proper length of spiral: Length of 6-chord spiral in feet = elevation of track in inches × speed in miles per hr. Thus for a 6° curve with elevation of 6 ins., and assumed speed of 40 miles per hr., length of 6-chord spiral would be 6 × 40 = 240 ft. The terminal curve (3°) would be ⅔ (240) = 160 ft. long.

AREAS OF WATERWAYS FOR CULVERTS AND SMALL BRIDGES.

There are two general methods for ascertaining the proper dimensions of waterway for culverts. One is based on the rainfall, area, etc., and the other on high-water marks and the general appearance of the waterway. Information as to high water is always desirable, when you can get it; the trouble is that, especially in country districts and sparsely settled regions, it is often impossible to find any reliable high-water marks.

As a general thing it would seem that, outside of cities, there are only three things that need to be seriously considered in determining, approximately, the quantity of water reaching a culvert, viz., the rainfall, the area drained, and the general slope of the area. The proportion wooded or cultivated need not generally be considered, as these are usually more or less temporary conditions. Some of the rougher regions in the mountains may always be covered with woods or brush, but such a condition would tend to reduce the run-off, and by disregarding it one would be on the side of safety. If it appears reasonably certain that the wooded condition is permanent, then, of course, some reduction in the size of opening would be justifiable. The shape of the valley might also affect the run-off; that is, it might diminish it where the valley is long and narrow; but the proposition is not to get a rule that shall be mathematically exact, but something that shall be sufficiently accurate for ordinary purposes. It may occasionally happen, however, that the fall of stream, below the culvert, is so small that the water will not run off, even though the culvert has ample area to discharge the water at a reasonable velocity. Practically a condition of backwater will then exist. This trouble will, of course, not be due to the size of culvert. It is then a question of high water; and the engineer can make his plans accordingly.

The importance of the high water feature increases with the size of the culvert or bridge; but a proper area for waterway is none the less necessary. In an interesting article by E. C. Murphy, published in Water Supply and Irrigation Paper No. 147, U. S. Geological Survey (1904), entitled "A Method of Computing Cross-section Area of Waterways," a table for discharge of streams, having a drainage area of 10,000 sq. miles and less is tabulated, but it does not seem well adapted to small areas, say under 10 sq. miles.

To determine within reasonable limits the proper size of a culvert from rainfall, area and slope of ground, would seem to be a comparatively simple matter. Table X. of minimum cross-section areas should be understood as applying only to the east half, or well watered portion of the United States. The run-off, or discharge, has been calculated by the well-known Bürkli-Ziegler and McMath formulas. The latter is used for larger areas, and the former for smaller and steeper ones where it gives somewhat larger values. The maximum flood velocity through the culvert is uniformly assumed at 6 ft. per sec. for the culvert running full. In case the opening were one-third filled with drift, the velocity would be 9 ft. per sec. These dimensions are suggested as about the proper thing for railroad culverts outside of cities. The writer is inclined to think that under favorable circumstances culverts of concrete or cement masonry with a good height of solid embankment above them might be used with the openings for waterways about one-third less than those given in the table, and still be reasonably safe. But if the embankment is sandy, or there is very little filling over the culvert, it will not do to take chances.

For small openings and culverts where it is simply a question as to the proper area of opening to carry the maximum discharge or "run-off" from the drainage area, we can proceed as follows: First get the area drained, from U. S. topographic sheets, by pacing, or by approximate measurement. Then take approximately the difference in elevation between the dividing ridge at the head of the valley and the ground at the mouth of culvert; also the distance from top of the divide to the culvert measured or estimated along the watercourse. Divide the difference in elevation by this distance, to get the slope or general fall of area in feet per thousand, and take from Table X. the required area of opening in square feet. It is not necessary to get the slope accurately, as a rough estimate or approximation will do, since the size of opening only varies according to the fourth or fifth root of the slope, and dividing the slope by two will only diminish the area of openings by 13% to 16%.

Table X. has been calculated for [illegible] from 2½ acres up to 100 sq. miles, but for [illegible] openings and bridges it should be understood to apply only under certain limiting conditions. Otherwise it should not be used except as a means to ascertain the discharge (six times the opening in square feet). It seems that the principal conditions which limit the application of a table of this kind to larger areas are the fall of stream below the opening, the velocity at the opening and the elevation of high water. The last two depend largely on the first.

The fall below the bridge and consequent velocity may be so great that a smaller opening would answer. If there is not enough fall below the bridge to give the necessary velocity through the opening of given size, the water will "bank up" somewhat on the upper side of bridge. In most cases, it would seem, this "banking up" of the water would be hardly appreciable; but the increased velocity and consequent liability to scour may be a more serious consideration.

The following examples will illustrate:

Take an area of 25 sq. miles in a mountainous section; having a general fall of 35 ft. in 1,000 ft.; a fall of 5 ft. in 1,000 ft. below the bridge; channel at governing points about 60 ft. wide in bottom, side slopes 1 to 1; and high-water mark uncertain, but apparently about 7 ft. above the bottom. The area, 705, given in Table X., multiplied by 6, gives 4,230 cu. ft. per sec., as the maximum discharge or run-off. By Kutter's formula (p. 564-568, Trautwine, 1902), assuming hydraulic radius at 5½ ft., and with .035 coefficient of roughness, we get velocity = $c\sqrt{RS}$ = 9.6 ft. per sec. (See also table, p. 191, in Water Supply and Irrigation paper mentioned above.) Then 4230 ÷ 9.6 = 440 sq. ft., the calculated cross-section area of flood waters (EFCD, Fig. 3 A). A water area of 440 sq. ft. would fill the channel to a depth of 6.6 ft., and give a hydraulic radius of 5.6 ft., which checks approximately the high-water mark and hydraulic radius assumed. Taking a clear span of 60 ft. at right angles to the current, and 705 sq. ft. area of opening as given in Table X., we would have an opening 60 × 11.7 ft. (ABCD, Fig. 3 A). An opening 60 × 11 ft. (abCD, Fig. 3 A) would probably be sufficient.

Again take the same area, slope, discharge and cross-section of channel, but with a fall of only ½ ft. per 1,000 ft., below the bridge: Figuring as above, we find high water will be 12.8 ft. deep, hydraulic radius 9.7 ft., velocity 4.55 ft. per sec., and maximum water area 932 sq. ft. (Fig. 3 B).

Table X. gives 705 sq. ft. opening, based on a velocity of 6 ft. per sec. running full, but if the water is not to fill the opening the velocity will have to be greater. Taking velocity at 8 ft. per sec, the water area will be ⅛ of the discharge or 4,230 ÷ 8 = 529 sq. ft. As the depth is 12.8 ft., the width will be 529 ÷ 12.8 = 41.3 ft. corresponding to the area, mnCD, Fig. 3 B. The total opening will have height (AD Fig 3 B)

$$= \frac{705}{41.3} = 17 \text{ ft.}$$

corresponding to ABCD, Fig. 3 B. The velocity in the unobstructed channel being 4.55 ft. per sec., the water will "bank up" on the upper side of bridge until it acquires a velocity head equivalent to (8 — 4.55) = 3.5 ft. per sec.

By formula, p. 516, Trautwine (1902), we in-

fer that this rise would be $\frac{V^2}{64.4} = 0.19$ ft., or by formula, p. 576 of same, $V^2 \times 0.017 = 0.21$ ft. In other words, the water would have to rise 0.2 ft. on the upper side of bridge. This would not seem to be serious; but as it may be objectionable to contract the waterway and increase the velocity very much, it might be well to make the span somewhat longer, say 50 ft. (See abcd, Fig. 3 B.) The velocity would then be $\frac{4.230}{50 \times 12.8} = 6.6$ ft. per sec., and would require a velocity head or rise on the upper side of the bridge by same formula of $2^2 \times 0.017 = 0.07$ ft.

A DIRECT READING NOZZLE PIEZOMETER.

By F. B. Sanborn,* M. Am. Soc. C. E.

A device has recently been developed by the writer for measuring the pressure and quantity of water that is being delivered by a fire stream or other flowing jet. This device, which is called a nozzle piezometer, is applied to the end of a nozzle as shown by Figs. 1 and 2. This can be done without shutting off the stream or making any fixed attachment, it only being necessary to hold the piezometer at the nozzle long enough to observe the reading. I have thus frequently read in ten minutes' time four streams scattered about a mill yard.

The method formerly employed for measuring the discharge through the nozzles, as used in tests of public water-supplies, pump or steam fire engines, has been to drill and tap a hole in a hydrant and take the reading of a gage at this hydrant as an average of the four or six that are in use. Knowing this hydrant pressure, reference would be made to Freeman's Fire Stream Tables, and by allowing for the quality of hose the amount flowing through a given nozzle would be computed. This method, as will be seen, involved a little trouble in rigging up apparatus and in computing results, which together have hindered the adoption of a nozzle as a common method of metering.

The aim of the present device is to do away with the fittings or attachments and the necessity of using printed tables. As represented in the illustration, the device consists of a portion called a piezometer and a pressure gage. The piezometer is kept in place by a shoulder that rests against the bead at the end of the nozzle, but it can be used on a smooth tip or even on a 2½-in. hose or hydrant outlet by taking a little pains to keep it in place.

The pressure is transmitted to the gage by a form of Pitot tube; but instead of an exposed tube, a passage is provided through a thin piece of metal, or "divider," that is ⅛ in. thick at the center and tapers to knife-edges—thus to enter the stream and receive the pressure without seriously interfering with the flow. A semi-circular shield extends over the end of the nozzle, covering the piece of metal, or divider, that enters the stream. Besides preventing the water from spraying or spattering, this shield serves the important function of keeping the divider within the stream—it being very difficult to hold even a thin piece of metal within a jet that is flowing with high velocity. The whole device, as constructed, however, is very easily held, merely pressure from the hand being sufficient to steady the gage, and prevent it from tipping over.

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Fig. 1. Applying a Nozzle Piezometer to a Hose Stream.

The gage gives effective pressure. That is, the full pressure that the stream possesses by virtue of the velocity that it has the instant it is leaving the nozzle. Freeman's Fire Stream Tables give the nozzle pressure as taken at a 2½-in. piezometer coupling placed at the base of an underwriter play pipe. For any given pressure, therefore, those tables show slightly greater discharges than does the following table, which gives the quantities of discharge for three common sizes of nozzles, as determined by the nozzle piezometer.

Pressure in lbs. per sq. in.	Discharge in gallons per minute. 1-in smooth nozzle.	1⅛-in. smooth nozzle.	1¼-in. smooth nozzle.
5.....	66	84	105
10.....	92	116	143
15.....	111	142	173
20.....	129	165	202
25.....	144	184	226
30.....	159	202	248
35.....	171	218	267
40.....	183	233	286
45.....	194	247	303
50.....	205	261	320
55.....	215	273	336
60.....	225	285	351
65.....	234	296	365
70.....	243	307	379
75.....	252	318	393
80.....	260	329	406
85.....	268	339	418
90.....	276	349	430
95.....	283	359	442
100.....	291	368	453

Since the pressure that is measured is the effective pressure of the stream as it leaves the nozzle, it becomes possible to graduate a gage that will indicate both pressure and discharge. Fig. 2 shows indistinctly a form of graduation that I have designed, and that has been made by the American Steam Gage & Valve Mfg. Co., of Boston. The face of the gage gives pressure per square inch as usual, but in addition the discharge in gallons per minute for three sizes of nozzles is given; in this case 1 in., 1⅛ and 1¼ in., and for a pressure of 10, 20, 30 and 100 lbs. The back of the gage gives quantities for pressure of 5, 15, 25 to 95 lbs. Thus by interpolating between the 5-lb. quantities, results can be obtained to the nearest pound.

As to the accuracy of this device, I have found that the feature to be guarded against is too large a reading; as, for instance, it will be evident that too large a reading would be obtained by constructing the divider or Pitot tube so as to block free delivery of the nozzle. There is small likelihood of obtaining readings that are too small. The range of the readings that can be obtained with the device in different positions lengthwise of the stream is about 3 lbs. for an ordinary pressure.

As to different positions sidewise: At the very edge of the stream a very low and plainly false reading will be obtained, but anywhere within the body of the jet a maximum and true reading. This is in accord with the results obtained in the careful experiments of Mr. Freeman. (Trans. Am. Soc. C. E., 1889.)

The necessity of making the divider, or Pitot tube, of thin construction so as not to obstruct the flow and cause undue spattering, has led me, as a result of numerous tests, to reduce the size of passageway at the entrance to 1-16 in., which is the size used for ordinary pressure gages; and, as in the case of pressure gages, it is possible when testing with muddy water to have this small passageway become obstructed. In using a piezometer three or four times a day for three weeks with the muddy waters of Virginia and West Virginia, I have had an obstruction occur once; a fellow-inspector using one in Massachusetts had it occur at the third test. To provide against possible annoyance from this source the parts are so constructed that the whole can be taken apart and only in exceptional cases will an obstruction occur that cannot be released by a pin or small piece of wire.

Fig. 2. Detail View of Nozzle Piezometer and of Gage Reading, Both Pounds and Gallons.

This method of gaging the flow of fire streams is applicable to ordinary tests and besides for observing the pressure and quantity of streams at the time of a fire. A district chief or a water-works inspector could make such a test of any stream without shutting off the water to attach a gage and even without interfering with the flow for more than a few seconds.

DUST PREVENTION EXPERIMENTS in Surrey County, England, are described in a recent report made by Mr. Frank G. Howell, County Surveyor. An appropriation of about $10,000 was made available last year for the purpose indicated. A number of sections of road in various parts of the county were used for experiments with (1) tar macadam in place of existing ordinary macadam, at a cost of about 4s. (97 cts.) per sq. yd.; (2) tar macadam laid on existing macadam, at a cost, according to thickness, of 1½s. to 3s. (36 cts. to 73 cts.) per sq. yd.; (3) painting or brushing macadamized roads with hot tar, after the dry road surface had been thoroughly swept; and (4) sprinkling road surfaces for two consecutive days and at subsequent intervals with a solution of calcium chloride, given the trade name of "Akonia." Brief notes on the several experimental stretches of road and the results obtained are given in the paper and may be found in the London "Surveyor" for Aug. 17, 1906. Mr. Howell's conclusions, as stated by the "Surveyor" are as follows:

A length of road tarred during exceptionally fine and dry weather may prove most successful, while another length where the work was interrupted by rains may cause many complaints and give much trouble before the surface will bind at all. It is still too early to speak with any certainty as to the cost of tar-macadam compared with that of ordinary macadam, as some years must elapse before the life of the tarred material can be ascertained, but I am of opinion that although this material is successful in some places where the traffic is light, it is quite unsuitable for the main roads in the county carrying the heavier traffic. Even upon the roads with light traffic the tar-macadam would have to last from eight to twelve years without renewal to bring the cost down approximately to that of ordinary macadam, and I feel sure that any attempt to use it on an extensive scale must result in a very large increase in the cost of maintenance. Unless carefully cleansed when slightly wet the surface becomes slippery. It is not dustless, and the dust from it is, in my opinion, of a more objectionable character than the dust from an ordinary road.

With regard to the experiments in painting the roads with tar, the initial expense is small, and a considerable amount of dust is prevented. Upon wind-swept, open roads the surface is preserved by the tar, and the cost of the tarring is practically saved. On granite roads near villages and towns the cost is not saved by the decreased wear, but the dust is lessened. The tarring can only be properly done during perfectly fine weather, and, unfortunately, however carefully the work is executed, the surface is liable to churn up during the heavy rains of the early winter, when the water seems to get under the tar and loosens the surface. At such times complaints are received of the inconvenience caused by the black mud, but it is usually found that the surface settles again after a few days of dry weather. I estimate that the tarred roads are upon the average in a bad condition for about 14 days during the winter, but in spite of this disadvantage, in many places the inhabitants along the roads are anxious for a renewal of the tarring in the summer to mitigate the intolerable nuisance from the dust caused by motor cars.

LOWERING THE TUNNELS UNDER THE CHICAGO RIVER.

The city of Chicago has three tunnels under the Chicago River, and all these are about to be lowered so as to give a depth of 26 ft. of water (instead of 17 ft.) in the river, as required by the Federal Government in connection with its river and harbor improvement works. Two of these tunnels, at Washington St. and La Salle St., were built by the city in 1866-1871 for street traffic, but were afterwards turned over to the street railway companies. A bridge was subsequently provided at Washington St.; there is no bridge at La Salle St., but the old subway for pedestrians has been maintained in use until recently, though it was not much used. The third tunnel, at Van Buren St., was built in 1889-1890 by a street railway company. The question of lowering the tunnels has been greatly complicated by the litigation and prolonged controversies between the city and the railway companies, but the government has insisted that the work must be done without further delay, and the Chicago Union Traction Co., which has succeeded the original companies, has finally agreed to do the work. The receivers of the company awarded contracts for the execution of the work on July 23, and it must be completed before the opening of navigation in 1907. The plans as adopted, and approved by the City Engineer on behalf of the city authorities, have been prepared by Mr. Samuel G. Artingstall, M. Am. Soc. C. E., Chief Engineer for the company. These plans, which are described below, provide for a new roof system of transverse beams and jack arches. The city engineer had prepared plans for a reinforced-concrete roof construction, but those of Mr. Artingstall were preferred by the company and approved by the city engineer.

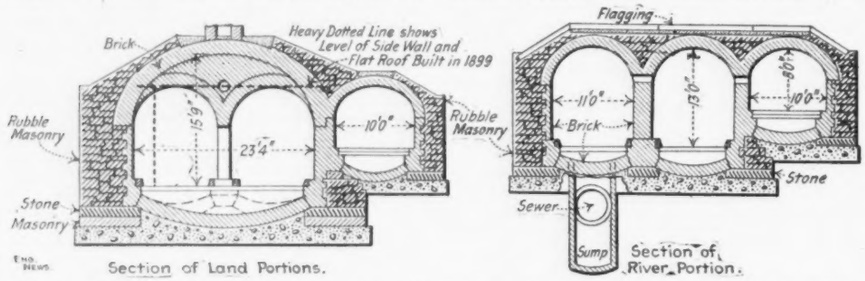


FIG. 1. ORIGINAL SECTION OF THE WASHINGTON ST. TUNNEL UNDER THE CHICAGO RIVER, CHICAGO.

It may be noted that only the Van Buren St. tunnel is to be kept available for traffic at the present time. The Washington St. and La Salle St. tunnels will be temporarily abandoned; the new roof will be put in to allow of removing the old roofs and deepening the river, but the floors will not be lowered, so that these two tunnels will not be available for traffic. The street railway traffic will be provided for, however, by stringing trolley wires along parts of the cable lines, and operating these lines by electric cars which will cross the river bridges. The change was in fact the final deathblow of the cable traction system in Chicago. It has long been foreseen that cable must be superseded by electric traction, but in consequence of the controversies between the city and the companies the former has refused, until recently, to permit the erection of poles and trolley wires along the cable lines. This conversion to the electric system is now partially completed, and it is expected that cable traction will be entirely abandoned by the end of the year.

In connection with the description of the methods of lowering the tunnels, it will be of interest to give some particulars of their history and construction. As long ago as 1853, a company proposed to build an iron-lined river tunnel according to plans prepared by the late Thomas C. Clarke, M. Am. Soc. C. E., but the more general use of swing bridges about that time led to the abandonment of this project, although it was followed by several others. In 1864, the Pittsburg, Fort Wayne & Chicago Ry. petitioned for the construction of a free tunnel at Adams St., in order to provide a connection with its terminal station on the west side of the river; the railway offered to pay $25,000 towards the cost, or $15,000 towards that of any tunnel not farther north than Washington St. Various propositions were made and locations suggested, but the Washington St. location was eventually selected by the city.

WASHINGTON ST. TUNNEL.

ORIGINAL TUNNEL.—In 1865, the city council passed an ordinance authorizing the construction of a tunnel at Washington St., and appropriated $100,000 in city bonds, provided that a similar amount could be raised by private subscription. Preliminary plans for this were prepared by Mr. Gindele, C. E., then president of the board of public works. The public subscription failed, and eventually the city passed another ordinance providing for the construction of the Washington St. tunnel by the board of public works. In August, 1866, there were 19 bids received, ranging from $247,000 to $430,000, and the contract was let to Stewart, Ludlam & Co., at $271,646. Work was commenced in October, but owing to lack of funds the contractors were unable to carry it out, and they abandoned it in May, 1867; in June, the abandoned excavation east of the river caved in. New plans and specifications were made and in July a new contract was awarded to J. K. Lake (associated with him being C. B. Farwell and J. Clark) at $328,500. The tunnel proper was built in open cut, half its length being built at a time in a cofferdam 150 ft. wide and extending about 110 ft. into the river. The cofferdam construction consisted of two rows of piles and sheet piling, connected by iron rods, and having the 15-ft. space between the rows filled with clay. Considerable trouble was experienced from vessels striking the cofferdams and causing leaks.

The tunnel consisted of three passageways; the south one being for foot passengers (and reached by steps from an entrance house in the middle of the street near the river) and the others being for horses and vehicles. The footway tunnel was 10 × 10 ft., but the headway above the plank flooring was only 8 ft. [illegible] other passages were 11 ft. wide and 15 ft. [illegible] high, or 13 ft. high from the paving, and [illegible] arated by a 2-ft. wall with arched openings [illegible] wide. There were similar openings in the [illegible] between the footway and the first roadway [illegible] tunnel. The walls, arches and inverts were [illegible] brick, laid on a concrete base, and having stone [illegible] backing and stone filling between the arches. [illegible] the river bottom was paved with 10-in. stone [illegible] flagging laid in concrete on top of the filling. This form of section continued for 222 ft., [illegible] a little beyond the bulkhead lines of the river. Beyond this, at each end, the two main passageways converged to a single arched passageway 23 ft. 4 ins. wide, gradually diminishing in 40 ft. to a width of 18 ft. 6 ins.; the height was 20 ft. 6 ins., or 15 ft. 9 ins. above the paving. The 10-ft. and 11-ft. arches in the river section had five brick rings; in the shore sections the 23-ft. arches had six rings at the crown, and the 10-ft. arch had four rings. The arches were covered with asphalt mastic mixed with 5% of prepared mineral bitumen; this was ½-in to ⅝-in. thick, laid on a bed of lime mortar and covered with ⅝-in. of mortar. The leakage at first was considerable, but was gradually stopped by caulking, etc. Two large cracks developed, extending entirely across the masonry and from top to bottom. These were due to the creeping of the clay towards the open space in the west cofferdam, but only one of them was under the river. The annual report of the City Engineer, Mr. Chesbrough, for 1869, describes this tunnel and states that pains were taken to prevent leakage through the roof by the use of asphalt prepared in the form of "government mastic" at Governor's Island, New York, under the direction of the U. S. Engineer Corps, and sold to the city at cost by the War Department. Unfortunately it was necessary to use it in frosty weather, on a frequently frozen bed.

The general design is shown in the cross sections in Fig. 1. The total length of the tunnel and approaches (from Franklin St. to Clinton St.) was 1,608 ft., while the footway tunnel was 811 ft. long. Under the river the grades were 1 in 42.86 on each side to the center, for all three passages. The approach grades were 1 in 16 on the east and 1 in 18.62 on the west side; except that in the footway tunnel they were 1 in 11.30 and 1 in 12.07, respectively. The total cost to the city was $512,707, of which $395,664 was paid to the contractors. The tunnel was opened on Jan. 1, 1869.

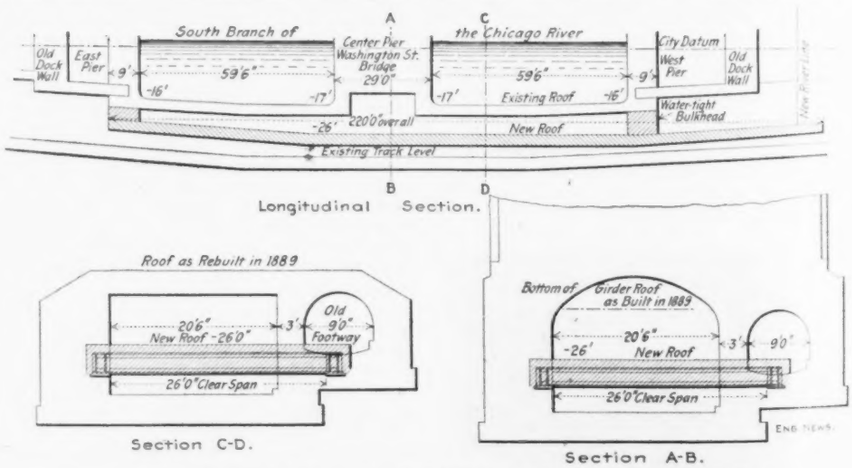
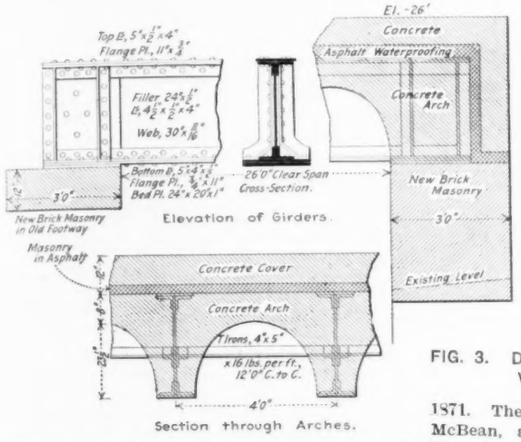


FIG. 2. NEW ROOF CONSTRUCTION FOR WASHINGTON ST. TUNNEL.

ALTERATIONS OF THE TUNNEL: 1889.—The work of lowering the tunnel now to be undertaken will be the second operation of this kind to which the tunnel has been subjected. In 1889 the city gave the West Chicago Cable Ry. Co. the right to use the tunnel (which was then somewhat dilapidated), one condition being that the roof should be lowered 3 ft., so as to give 17 ft. instead of 14 ft. of water in the river. This

work was done within cofferdams, only half of the river being obstructed at a time, as described in our issue of Dec. 13, 1890. In the reconstruction, the center dividing wall was removed, a new 36-in. brick wall was built between the footway and the roadway, and a new 54-in. wall built against the inside of the old 60-in. masonry wall on the north side. The width of the tunnel as reconstructed was 20 ft. The new roof consisted of 20-in. steel I-beams 24 ft. long, spaced 3 ft. c. to. c., with brick arches between them covered by 10 ins. of concrete. The arches had a rise of 8 ins. and were built with three rings of brick, the outer ring



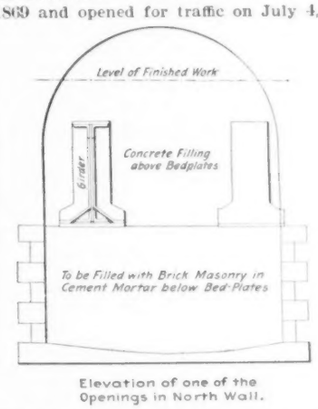

Level of Finished Work
Girder
Concrete Filling above Bedplates
To be Filled with Brick Masonry in Cement Mortar below Bed-Plates
Elevation of one of the Openings in North Wall.

FIG. 3. DETAILS OF ROOF CONSTRUCTION; WASHINGTON ST. TUNNEL.

being laid in asphalt. The floor was lowered 2 ft. 6 ins. to give the necessary headway for cars. As the reconstructed tunnel would not accommodate street traffic a drawbridge had to be built across the river, the center pier resting upon an arch of 20 ft. span and 6 ft. 6 ins. rise, taking the place of the beam roof. This was not built in open cut, but the old roof was temporarily left undisturbed at this point and the new arch built inside the old roof. This work was done by FitzSimons & Connell. The old footway tunnel is now only used as a pipe subway, and the entrances (in the middle of the street) were, of course, removed when the street was continued to the bridge.

ALTERATIONS OF THE TUNNEL: 1906.—The arrangement and character of the new roof construction is shown in Fig. 2. It will be seen that in the middle portion of the tunnel the new roof will be only about 2 ft. 6 ins. above the present floor. The footway (or present pipe subway) will be entirely destroyed, and the openings between this and the main tunnel will be closed with concrete. The length of the new roof will be 220 ft., and at the pier lines water-

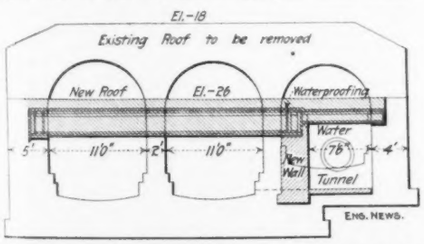


Fig. 4. Section of the La Salle St. Tunnel Under the Chicago River, Showing Portion of New Roof.

tight bulkheads will be built to hold back the water when the existing roof is destroyed. As already noted, no plans have been made for lowering or reconstructing the tunnel floor; the new roof will be built and the old one removed, and then the tunnel will be abandoned awaiting further developments. The construction of the new roof is clearly shown in Fig. 3; transverse plate girders 30 ins. deep and 4 ft. apart will be supported on new brick masonry in the tunnel wall and the old footway, giving a clear span of 26 ft. between the walls. Concrete arches, 8 ins. thick at the crown, will be built between the girders, and leveled off flush with their top chords. Upon this concrete will be 3 ins. of masonry in asphalt and 9 ins. of concrete. When the lowering of the floor is eventually undertaken, the new approach grades will be about 9.33% (instead of 7.58%) on the east, and 7.87% (instead of 6.33%) on the west side. The contract for the work has been awarded to Angus Bros. & Co., of Chicago.

LA SALLE ST. TUNNEL.

ORIGINAL TUNNEL.—This tunnel was commenced in 1869 and opened for traffic on July 4, 1871. The contractors were Moss, Chambers & McBean, and the total cost to the city was $566,000. The general type of construction was very similar to that of the Washington St tunnel (Fig. 1), with a few modifications: the footway was made 12 ft. high, with a clear height of 10 ft. (instead of 8 ft.) above the pavement, and the steepest grades were 1 in 20 instead of 1 in 16. In the roof, the outer courses of brick were laid in asphalt, instead of being simply covered with a layer of asphalt. The total length was 1,890 ft., of which 300 ft. was arched work under the river, 870 ft. arched work under land on either side, 174 ft. of flat roof with transverse beams and jack arches, and 546 ft. in open cut on the inclined approaches. This tunnel has not been altered since its construction, and the footway tunnel is still open to pedestrians.

ALTERATION OF THE TUNNEL: 1906.—This work will be very similar to that of the Washington St. tunnel, already described, except that the old footway will be retained and used for a pipe tunnel, as shown in the cross section, Fig. 4. The main roof construction will be the same as in the Washington St. tunnel, and the girders will rest upon the old west wall and a new brick wall built inside the footway. To one end of each girder, however, will be riveted a 12-in. I-beam extending across the pipe subway; a flat concrete roof will be built between these beams, being reinforced by steel bars laid between the bottom flanges and over the top flanges of the beams, as shown in Fig. 5. When the floor is lowered, the new grades will be 5.89% (instead of 4.32% and 4.95%) on the east, and 6.44% (instead of 4.83% and 5.00%) on the west side. The contract for the work has been awarded to John P. Agnew, of Chicago.

VAN BUREN ST. TUNNEL.

ORIGINAL DESIGN.—This is the latest of the Chicago River tunnels, having been built in 1889-90 by the West Chicago St. Ry. Co., of which Mr. Samuel G. Artingstall was chief engineer. Its construction was described in full in our issue of Feb. 22, 1890. As shown by the typical cross section in Fig. 6 it is in the form of a flat horseshoe arch, 30 ft. wide, and with a headway of 15 ft. above the flat floor, or 20 ft. above the concrete invert. The crown has a radius of 16 ft., and the sides have a radius of 11 ft. At the level of the floor are lines of heavy masonry blocks forming the skewbacks for the 7-ring arch; the three outer rings were laid in asphalt-cement mortar, composed of 1 part Trinidad asphalt to 3 parts of gypsum. Under the river, also, the concrete filling was covered with 1 in. of this asphalt mortar, on which was laid a paving of flag-stones 12 ins. thick. The work was done in an open cofferdam, only half the river being obstructed at one time. The contractors were FitzSimons & Connell. The tunnel proper is 924 ft. long (of which 220 ft. are under the river), with open inclined approaches aggregating 600 ft., making a total length of 1,524 ft. The grades are 10% at the east, and 5.46% and 8% at the west side.

RECONSTRUCTION OF TUNNEL: 1906.—This tunnel is not only to have the roof rebuilt at a lower level but is to be entirely reconstructed so as to be kept open for traffic. The original and new levels are shown on the profile in Fig. 7, while Fig. 8 shows the character of the reconstruction work. The side walls will be cut away and the massive stone skewbacks leveled to form seats for columns built of 15-in. I-beams, which will support 32-in. roof girders spaced 4 ft. 3 ins. c. to c. of webs. The ends of these girders are to be cut at an angle so as to reduce the amount of excavation of the old masonry. Between the girders will be longitudinal tee bars to form supports for the trolley wires (the cable traction system being abandoned). Upon the tops of the girders will be laid convex buckle plates, riveted to the top chords, and over these will be concrete filling. In order to provide against leakage, lead calking joints are used, as shown in Fig. 9. A lead filling will be placed between the buckle plates and the girders, and the joints can be calked by a hammer and chisel wherever leakage may develop.

For the lower part of the tunnel the old work will be underpinned at the sides by concrete, forming new side walls, and a new concrete invert will be built, as shown in Fig. 7. By shortening the length of the level portion under the river the grades will not be materially changed, being 11.06% on the east and 6.32% on the west side. The contract for the work has been awarded to the Great Lakes Dredge & Dock Co., of Chicago.

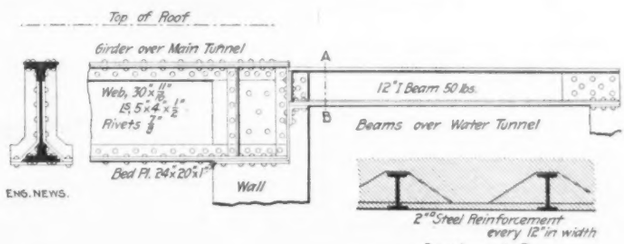


FIG. 5. DETAILS OF ROOF CONSTRUCTION; LA SALLE ST. TUNNEL.

SPECIFICATIONS FOR RECONSTRUCTION WORK.

For the work on the two tunnels owned by the city (at Washington St. and La Salle St.) specifications have been prepared by Mr. John Ericson, City Engineer, and these must be observed in the execution of the work by the contractors for the street railway company. The specifications state that the new roofs must give a depth of not less than 26 ft. below datum (mean water level) in the channel and 23 ft. at

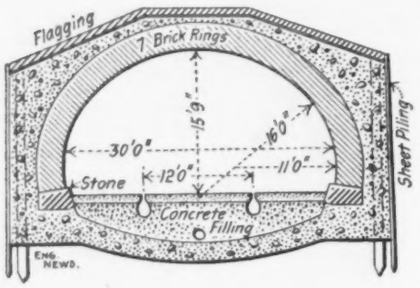


Fig. 6. Original Section of the Van Buren St. Tunnel.

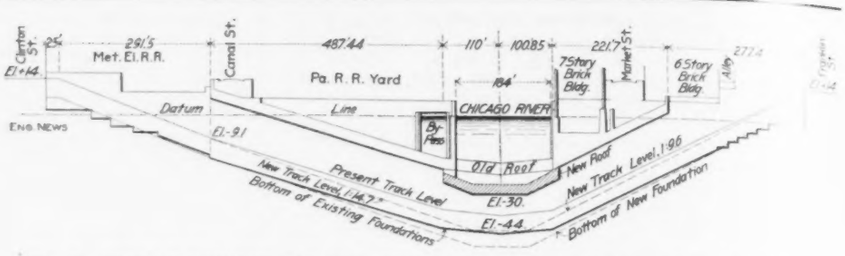


FIG. 7. PROFILE SHOWING OLD AND NEW CONSTRUCTION OF VAN BUREN ST. TUNNEL.

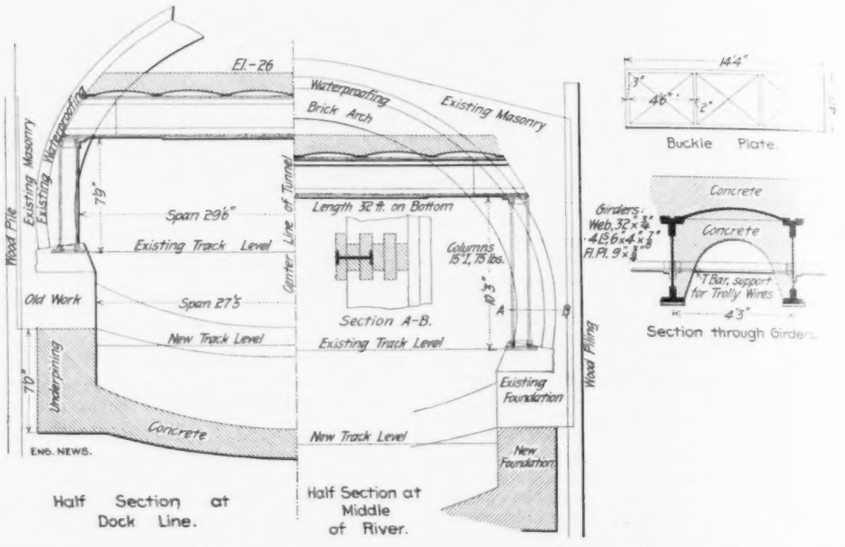


FIG. 8. CROSS-SECTIONS OF THE VAN BUREN ST. TUNNEL, SHOWING METHODS OF CONSTRUCTION.

the dock lines. Some extracts from these specifications are given below, and it may be noted that the cement used is required to comply with the specifications recommended by the American Society of Civil Engineers:

CONCRETE.—There shall be two grades of concrete. The one for filling and roof cover shall be composed of 1 measured volume of cement, 3 volumes of sand and 6 volumes of crushed limestone, which shall not exceed 2 ins. cube. The second grade, for use between girders, shall be composed of 1 volume of cement, 2 volumes of sand and 4 volumes of crushed limestone, not exceeding 1 in. cube.

All mortar for brickwork and for finishing surfaces shall be 1 part cement and 2 parts sand. All mortar for grouting shall be 1 part cement to 1 part sand.

All concretes and mortars shall be well mixed and tempered with clean water by a mechanical mixer. The concrete shall be rammed in layers 4 to 6 ins. thick, and shall be used before it has become hard and brittle by the setting of the cement. No re-tempering will be allowed in any case. In joining new concrete to old work the surface of the old work shall be cleaned, all loose stone or brick and foreign matter removed, the surface sprinkled with fresh grout immediately before laying the new concrete, to insure a thorough union between the old work and the new.

BRICKS.—All bricks shall be made from well-tempered clay, uniform in quality, sound and hard burned, free from lime, pebbles and cracks, with edges full and square, and of standard dimensions, namely, 8 × 4 × 2¼ ins.; all bats and defective bricks shall be rejected.

ASPHALT.—All asphalt shall be best Bermudez Lake asphalt and residuum oil in such proportions as designated by the engineer, so as to make a cement which shall be adhesive and plastic under the most unfavorable conditions, and mixed while hot with gypsum, in proportion of about 50% of gypsum. This shall be poured on the work where required to make a waterproof cover or shield. Where this cement is used all bricks shall be free from moisture, and the work (whether new or old) shall be perfectly clean and dry, and the existing work heated when possible so as to effect a perfect union between new and old work.

The waterproofing shall consist of brick laid flat the whole length and breadth of the river section and down the outer edge of the side walls. These bricks shall be embedded in asphalt mortar not less than ⅛-in. thick, and all joints in brick shall be filled and covered over the whole surface with asphalt mortar. No joint or cover shall be less than ¼-in. All asphalt shall be applied at a temperature of not less than 180° F. All moisture in bricks shall be expelled by heating to a temperature of not less than 90° before being used.

STEEL.—The steel may be either open-hearth or Bessemer steel; it shall contain less than 0.08% of phosphorus. It shall have a tensile strength of not less than 62,000 lbs. nor more than 70,000 lbs. per sq. in.; an elastic limit of not less than 35,000 lbs., with 22% elongation in 8 ins., and shall comply with all the conditions and requirements of the "Manufacturers'" standard specifications for structural steel of this grade. All steel work shall be thoroughly cleaned from all loose scale and rust before being embedded in the concrete.

CUTTING OLD MASONRY.—Openings shall be carefully cut in the existing masonry where required to allow the girders to pass through. Seats for girders shall be cut in the north wall and the masonry prepared for the bed plates of girders. All alcoves in this wall must be closed or built up with brick masonry in Portland cement mortar and thoroughly bonded into the old work. Seats for the ends of girders in the old footway must be carefully prepared by removing the existing old work the whole length of the river section to a depth of 12 ins. below the bed plates and built up solid with brick masonry and leveled for the bed plates. All possible precautions must be taken to avoid any settlement, disturbance or leaks in the old roof until the new work is finished, completed and found perfect.

BRICK MASONRY.—All brick shall be hard burned to a vitrified surface, laid in cement mortar in bond of alternate header and stretcher, all joints to be filled with mortar by pressing the bricks into the soft mortar, and not by forcing the mortar into the joints. No joints in the masonry shall be less than ¼-in. nor more than ½-in. thick. All bricks (unless otherwise specified) shall be immersed in water at least two hours before being used. All new work shall be thoroughly bonded into the old work, and all visible joints neatly struck and cleaned off.

CONCRETE COVERING.—The concrete cover shall be tamped in place in short sections, and the utmost precaution taken to avoid leaks or movements, and to make a perfect, solid and water tight junction with the old work.

ARCHES BETWEEN GIRDERS.—The arches between the girders shall be built on suitable centers and thoroughly rammed in place; care must especially be taken to insure perfect contact at all points with the steel work.

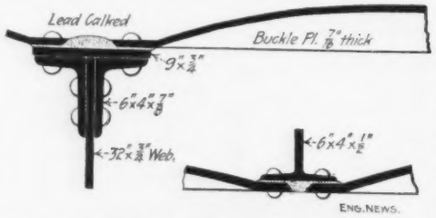


Fig. 9. Lead Calking Joints for the Roof of the Van Buren St. Tunnel.

The visible surfaces shall be finished with a smooth cement surface after the centering has been removed.

TAKING OFF THE OLD ROOF.—The existing roof of the tunnel and the center pier of the bridge* shall be removed after the new roof and new work have been finished and found perfect in every respect. The roof of the existing tunnel shall be destroyed as much as possible from the underside, then the space between the new roof and the old work shall be packed with clay or other suitable material, to act as a cushion; the old roof shall then be removed in such a manner as to avoid as much as possible any jars, tremors or concussions tending to injure the new work; it shall be done without the aid or use of cofferdams or explosives, and to interfere as little as possible with navigation in the river. The existing piles along the outer walls of the tunnel shall be cut off 12 ins. below the top of new roof, but the piles must not be drawn or loosened in any case. All debris or refuse shall be removed from the bed of the river to a level with finished work.

*At Washington St. only.

COMBINED RACK AND ADHESION LOCOMOTIVES have recently been built for the Benquella Ry., which is now being constructed from Benquella, on the west coast of Africa, to Caconda, about 200 miles. The road is ultimately expected to reach Katanga, 950 miles, in the Congo Free State. The railway is in Portuguese West Africa, in the province of Angola, but is a considerable distance south of the ambitious Portuguese Royal Trans-African Ry., which starts from St. Paul de Loando. The line is of 3½-ft. gage, with curves of 500 ft. radius and maximum grades of 2.5% on the adhesion section and 6.25% on the rack-rail section. The locomotives are tank engines of the 0-6-2 type, and are four-cylinder compounds, but only work compound on the rack-rail inclines. The cylinders are all the same size, and the two on each side are placed one above the other. All the cylinder have tail rods, and the valves are operated by a modified Walschaert valve gear. The lower cylinders are connected to the middle driving wheels and drive the engine as an ordinary simple engine on the adhesion sections. On the rack-rail sections, the exhaust is admitted to the upper cylinders, which drive a shaft with a pinion gearing with a spur wheel on the shaft which carries the single cog-wheel for the rack. Thus on the inclines the engines work both the adhesion and cog driving wheels. The rack is of the Marsh type (known in Europe as the Riggenbach type), and the rack-rail equipment and the locomotives were built by the Maschinenfabrik Esslingen, of Stuttgart, Germany. The contractors for the railway are Griffiths & Co., of London, England. The principal dimensions of the locomotives are as follows:

Driving wheels (6)	3 ft. 4 ins.
Trailing wheels (2)	2 ft. 5 ins.
Cog wheel (1), pitch diameter	3 ft. 1½ ins.
Wheelbase, driving	9 ft. 10 ins.
Wheelbase, total	16 ft. 3 ins.
Weight on drivers	25.78 tons
Weight, total	43.34 tons
Cylinders (4)	17¼ × 19 ins.
Boiler, diameter	4 ft. 2 ins.
Steam pressure	200 lbs.
Tubes; No., 187; length	4 ft. 10½ ins.
Heating surface; tubes	944.5 sq. ft.
Firebox	99.6 sq. ft.
Total	1,044.1 sq. ft.
Grate area	20.0 sq. ft.
Coal in rear bunker	1,600 lbs.
Water in side tanks	672 gals.

THE RELATION OF TRACK CONSTRUCTION TO SPEED AND WEIGHT OF TRAINS.

The relation of the construction of railway track to the character of the traffic which it is to carry has been discussed in our columns at different times, and it has been shown that taken as a whole the character of the track has not been strengthened or improved in due proportion to the increase in weight and amount of the traffic imposed upon it. The matter was discussed in a general way in our issue of Sept. 8, 1904, while in our issue of April 24, 1902, it was discussed at some length with special reference to engine loads and wheel loads. In the latter connection it must be borne in mind that the wheel loads of cars on most roads are now as heavy as the driving wheel loads of engines in general service ten or twelve years ago. The relation of track construction to speed of trains is another special phase of the matter which has more recently come up for consideration, and investigation appears to indicate that comparatively little attention is given to this matter. Equipment is distributed and schedules are made or revised without much regard to the character of the track construction, or to the questions of increased risk and of increased cost of maintenance. An inquiry as to conditions on a number of railways of different classes has brought forth some interesting information, which is presented below. The inquiry covered specially the question of the relative weight and stability of track on main lines where the traffic is (1) mainly freight with some slow or moderate speed passenger trains, and (2) both passenger and freight, including passenger trains running at a speed of 50 miles an hour or more. In other words, with a track of average construction for the former class, what changes would be required to put it in condition to safely carry the latter class? While no definite statement can be made on this point, the information given represents present day practice.

ATLANTIC COAST LINE RY.—On account of the character of the soil in this section of the country, 70-lb. rail with 16 ties to a 30-ft. rail and 8 ins. of gravel ballast would be first class track for slow traffic. For the fast heavy traffic, 85-lb. rails with 18 ties to a 33-ft. rail, and 12 ins. of ballast under the ties, is the generally accepted standard. There are 56-lb. rails on some of the lines which do quite a heavy business, but 70-lb. rails are the minimum on what are called the main lines. The heaviest engines operating over the lightest rails are of the ten-wheel (4-6-0) type, weighing 150,000 lbs., with 116,000 lbs. on the drivers. No consolidation engines are used on any of the trains. The maximum speed is 60 miles an hour and sometimes a little more; in fact, there is no speed limit on this very light rail. A large percentage of the business consists of passenger and perishable freight.

BALTIMORE & OHIO RY.—There are so many conditions affecting the answers to the questions that it would be very difficult, without naming each local condition, to state what is necessary to maintain track under the conditions mentioned. The weight of rail depends entirely on the weight of equipment; there are branch lines laid with 62-lb. rails which are carrying considerable traffic and are satisfactory. Other branch lines laid with 75-lb. rails carry heavy traffic. There are 16 ties under a 30-ft. rail and 18 ties under a 33-ft. rail in all main tracks. For fast traffic, the minimum weight of rail should be 85 lbs. per yd. If the traffic is exceedingly fast, with fairly heavy freight traffic, a really high-class track requires 100-lb. rails, with 16 ties under a 30-ft. rail and 18 under a 33-ft. rail. The minimum weight of rail on main lines is 75 lbs., and these are gradually being replaced with 85-lb. rails.

The heaviest locomotives operated are of the Pacific (4-6-2) type, weighing 229,500 lbs. There is no very accurate record of the speed over the light rail tracks, but in some cases the speed reaches 60 miles per hour. No investigations have been made as to the effect of increase of speed, weight of engines, and number of trains upon the wear of track and maintenance-of-way expenses, but these expenses have very materially increased during the past few years. There is a record showing the increase, but no comparison has been made to show how it has been affected by the increase in traffic. The freight traffic represents about 79% and the passenger traffic about 21% of the total traffic of this road.

CLEVELAND, AKRON & COLUMBUS RY.—The character of track depends upon the weight of equipment. For engines of 90 tons and upward and modern freight-car equipment, the minimum weight of rail should be 85 lbs., with 2,880 ties per mile (18 ties per rail length of 33 ft.), and the depth of ballast under the ties should be 10 ins. for the slow traffic or 12 ins. for the fast traffic. It is important to have a good roadbed for both the first and second classes of traffic, the difference being principally in the maintenance; that is, the track need only be maintained in ordinary condition for freight and slow traffic, whereas for high-speed traffic it must be maintained in first-class condition. The lightest rails are 70 lbs. per yd. The heaviest engines running on these rails are as follows: (1) Freight engines, 193,500 lbs.; weight on truck, 20,500 lbs.; weight on first pair of drivers, 43,500 lbs.; second pair, 41,500 lbs.; third pair, 46,000 lbs.; fourth pair, 42,000 lbs. (2) Passenger engines, 138,700 lbs.; weight on truck, 43,800 lbs.; weight on first pair of drivers, 48,100 lbs.; second pair, 46,800 lbs. The maximum speed for freight trains is 40 miles per hour, and for passenger trains 70 miles per hour. The percentage of freight traffic is 75%, passenger traffic 25%.

DULUTH & IRON RANGE RY.—For the slower class of traffic, if on branch lines and secondary main lines, the track should have 60-lb. rails and 18,000 lin. ins. of rail bearing on ties (measured on the rail seat); minimum depth of ballast under ties, 6 ins. For the fast traffic, the track should have 75-lb. rails, with 22,000 lin. ins. rail-bearing on face of ties, and 12 ins. of ballast under the ties. The size of the ties varies so much that the mere number per mile does not convey any definite meaning. The lightest rails on main track are 80 lbs. per yd., and the heaviest locomotives are of the consolidation (2-8-0) type, with 170,000 lbs. on the drivers. Passenger trains run at 30 to 50 miles an hour; ore trains, 10 to 45 miles an hour. Freight traffic represents 85.6% and passenger traffic 14.4% of the total. Maintenance-of-way expenses are increasing with speed, with weight of engines, and with number of trains. The effect of speed depends largely on the character of the traffic and the vehicles in which it is carried. For instance: A certain type of cars has a faculty of swaying or rocking to such an extent that beyond a certain speed they become absolutely dangerous, developing a tendency to climb the rail, owing to the magnitude of oscillations. While the total weight of engines is a factor, the distribution of the weight on the track is of a good deal more importance. The experience here fully bears out the investigations made by Mr. Dudley some two years ago on the effect of the distribution of wheel loads of locomotives on the structure of the track and the relatively high destructiveness to track of consolidation engines due to the large unloaded space of track between the front axle of the tender and the rear pair of drivers. The wear of track is fairly proportioned to the number of trains. The main distributing factor, however, is to be found, undoubtedly, in the unbalanced condition of reciprocating parts of the locomotives.

DULUTH, MISSABE & NORTHERN RY.—It is difficult to reply definitely as to proper track construction for different speeds, as the kind of traffic will largely influence the weight of rail to be used; this road uses 80-lb. rails of the Am. Soc. C. E. section. The heaviest locomotives in service are of the consolidation (2-8-0) type, with 165,200 lbs. on the driving wheels and 18,500 lbs. on the forward truck; cylinders 22 x 28 ins.; driving wheels, 4 ft. 8 ins. diameter. The maximum speeds are 30 miles per hour for freight trains and 40 miles per hour for passenger trains. Freight traffic is 98% and passenger traffic only 2% of the total.

HOCKING VALLEY RY.—Main-track construction for slow traffic comprises 60-lb. rails, with 2,680 ties per mile and 8 to 10 ins. of ballast under the ties. For fast traffic, 90-lb. rails should be used, with 2,800 ties per mile and 12 to 18 ins. of ballast under the ties. The service on the lighter track includes 83-ton locomotives of the consolidation (2-8-0) type, with 75 tons on the driving wheels, and the maximum speed over this track is 60 miles an hour. On this road 90% of the traffic is freight traffic.

NORFOLK & WESTERN RY.—While this inquiry is based upon two classes of traffic, the American Railway Engineering and Maintenance-of-Way Association is working under the principle of dividing railways into three classses, according to their traffic conditions. (Eng. News, March 29, 1906.) The track construction for these classes should be about as follows: Class A (50 miles per hour); 85-lb. minimum weight of rail, 2,880 ties per mile, and 12 ins. of ballast under the ties. Class B (40 miles an hour); 75-lb. rails, 2,880 ties and 10 ins. of ballast. Class C; 60-lb. rails, 2,560 ties and 6 ins. of ballast. The lightest rails in main track of this road are 75-lbs. per yd. (although no new rails are now purchased lighter than 85-lbs. per yd.), and over these run locomotives weighing 200,000 lbs. (without tender) and having maximum axle loads of 44,000 lbs.; the highest speed over these rails is about 60 miles an hour. On unimportant branches with 60 and 67-lb. rails, engines are used which weigh 138,000 lbs. (without tender) and having maximum axle loads of 36,000 lbs. From observation, investigation and records it is evident that all items of maintenance expenses increase with the density of the traffic; that is (1) largely with the total tonnage (and this is practically proportional to the total number of cars handled per mile), and (2) also to some extent with the speed of trains. The weight of engines affects curved track more particularly, and the wear of rails on such track; the wear of rails, however, is most generally proportional to the total tonnage passing over it. Passenger traffic on this road is about 33% of the total.

In regard to the number of ties, some roads handling a heavy traffic use, with entire safety and success, as few as 2,560 ties per mile, not only on portions of their road that can be graded as Class C, but also on higher classes. This good result is obtained by special regard to roadbed and the maintenance of good ballast. The Norfolk & Western Ry., however, is using about 2,880 ties per mile on all its road, and pays great attention to roadbed and ballast, especially to the improvement of roadbed. The most important feature about a railway required to handle dense or fast traffic is that it shall have a wide well-drained roadbed and good ballast. With these two features and with rails strong enough to carry loads there is no great variation in other items of construction.

SOUTHERN RY.—For slow traffic on main track there should be 80-lb. rails, ties spaced 24 ins. c. to c., and depth of ballast not less than 8 ins., but depending upon the character of the roadbed. For high speed traffic, the weight of rail should be 85 to 100 lbs. per yd., the ties and ballast remaining the same. Rails of 75, 80 and 85 lbs. per yd. are expected to be standard on this road, but at present some lighter rails are in use. The speeds over light rails are from 50 to 60 miles an hour.

UNION PACIFIC RY.—For both classes of traffic condition as mentioned it is the practice to use 18 ties to the 33-ft. rail and 8 ins. of ballast under the ties. With a large number of high-speed trains and excessive traffic, depth of ballast might be made from 10 to 12 ins. For high speed trains and heavy traffic, at least 90-lb. rails should be used, while with slower traffic on main lines lighter rails would in many cases be sufficient, say 75 lbs. The minimum weight of rail on the main track of main lines is 75

lbs.; there are also 80 and 90-lb. rails. Over this minimum weight of rail are operated freight engines with 187,000 lbs. on driving wheels, while passenger trains are frequently run over these rails at speeds of 55 to 60 miles an hour. No special investigation has been made as to the effect of increase in speed, weight of engines and number of trains upon the wear of track and the maintenance-of-way expenses, but experience shows in a general way that increased speed and weight of rolling stock results in considerable increased wear on rail and greater maintenance expenses. A general average of traffic on the above lines shows that about 60% is freight and 40% passenger traffic.

SOME CHARACTERISTICS OF COAL AS AFFECTING PERFORMANCE OF STEAM BOILERS.*

By W. L. Abbott.†

The capacity and efficiency obtained with a steam boiler is the result of many influences more or less variable in character, and for the purpose of studying some of these influences certain experiments were conducted, in which coal screenings were used, the results of which are presented in this paper.

The apparatus employed consisted of two Babcock & Wilcox boilers, one being 14 tubes high and 18 wide, of approximately 5,000 sq. ft. heating surface, fitted with a chain grate stoker of 75 sq. ft. area, which discharged the gases of the fire from under an ignition arch 5 ft. long, immediately among the tubes of the boiler. This boiler was also fitted with a Babcock & Wilcox superheater having an approximate area of 1,000 sq. ft. The other apparatus employed in one of the series of tests differed only in sizes; its boiler was 12 tubes high and 16 wide, contained 4,000 sq. ft. of heating surface, provided with a superheater and served with a chain grate stoker of 66 sq. ft. area.

The first experiment consisted in the use of coal separated into various sizes by means of screens having the following square openings: 0.25, 0.50, 0.75, 1.0 and 1.25 ins. The coal was all from one lot, so that the different portions resulting from the screening process were necessarily the same kind of coal, except that some portions were uniformly larger and others smaller, and that the smallest, on account of its size, was higher in ash. In all other respects, however, everything was equal; that is, influencing conditions, except that due to size of coal, were constant. In this way relative values were obtained, as shown by the curves of Fig. 1.

The percentage of ash in the different sizes of dry coal follows:

	Square screens. Through.	Over.	Ash in dry coal.
Sizes of coal in inches.....	1.25	1.00	13.7
" " "	1.00	0.75	14.0
" " "	0.75	0.50	15.6
" " "	0.50	0.25	20.8
" " "	0.25	0.00	30.8

The high per cent. in the smallest size is not due to ash in the coal itself, but to the fact that all of the fine sized foreign matter separated from larger coal, or which comes from roof or floor of the mine, naturally finds its way into this smaller coal.

Fig. 2 gives results of 18 tests made to determine the effect of varying quantities of ash associated with coal. One result of its presence is to reduce the heating power, owing to displacement of combustible matter. Therefore, in this connection, ash may be considered as a dilutant, and if this was the only result of its presence it would have no effect on heat efficiency secured through a boiler. A proportionately less amount of water would be evaporated by a pound of such mixture of ash and coal, of course, but efficiency would not be affected. If, however, ash acts in some other way as well, such as an obstruction to the combustion process, the effect of its presence is doubly harmful.

To insure that the result would not be affected by any influence other than that of the ash special coal was used. It was prepared in a Stewart washer and is known to the trade as No. 4 washed coal, a size made by passing over a screen having ¼-in. round openings, and through one having similar ½-in. openings. Its composition was as follows:

Moist coal—	
Moisture	7.48
Ash	8.23
B. T. U.....	12,191
Dry coal—	
Ash	8.90
B. T. U.....	13,176
Pure coal—	
B. T. U.....	14,463

The test showing the highest efficiency and capacity was made with coal just as it arrived, or, in other words, was of the composition shown by the analysis.

*Extracts from a paper read before the Western Society of Engineers.

†Chief Operating Engineer of the Chicago Edison Co., Chicago, Ill.

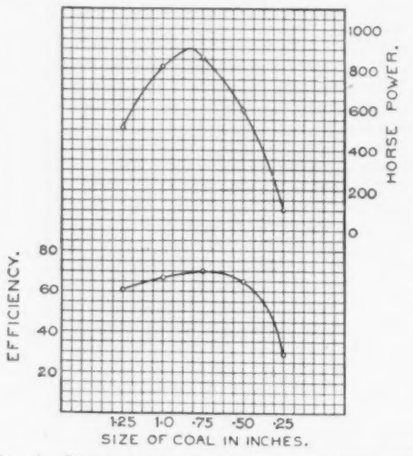


Fig. 1. Effect of Size of Coal on Efficiency and Capacity of a Boiler.

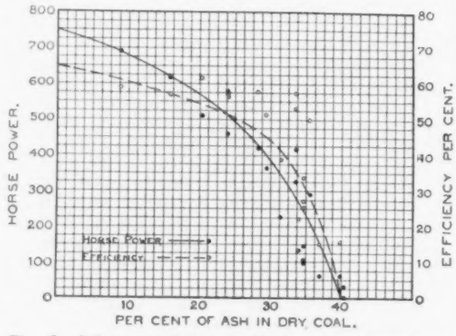


Fig. 2. Influence of Ash in Coal on the Capacity and Efficiency of a Boiler.

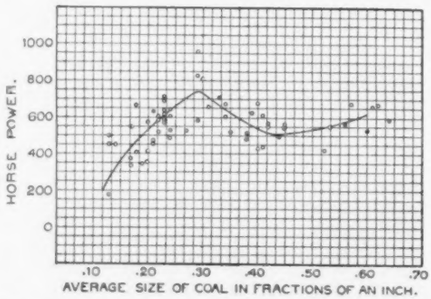


Fig. 3. Effect of Size of Coal Screenings on the Capacity of a Boiler.

Beginning with the tests of the second day, a quantity of refuse from the stoker ash pits was added to the coal to be used. This refuse was first weighed and the large pieces broken up, after which it was thoroughly mixed with the coal in the required proportion, and increasing amounts were added in each test which followed. This fuel composition was, of course, weighed as used, and a sample of it selected for analysis in the regular manner.

It appears from the curves (Fig. 2) that useful effect from the fuel drops to zero with 40% of ash, notwithstanding the fact that the other 60% of the composition was pure coal. The fact should be emphasized that although over half of the composition fed to the fire was fuel, it burned without producing any useful effect, for which there are two reasons—one, that on account of obstructed air supply through the fuel bed incomplete combustion and escaping hydrocarbons carried away a portion of the heat, because the gases passed immediately among the tubes of the boiler. The other is that owing to the presence of an excess of ash the percentage of fuel on the rear portion of the grate is greatly reduced. On this account a larger proportion of the air passing through the fuel bed does not combine with the fuel, but enters the furnace as free air. As the prime function of a furnace is to heat the gases passing through it, any increase in the amount of air entering the furnace without a corresponding increase in the amount of fuel burned must result in a lowering of the furnace temperature. This lowering of temperature, besides making a long, smoky flame which reaches up among the boiler tubes and is there chilled to below the burning point, also reacts on the fuel bed, reducing the rate of

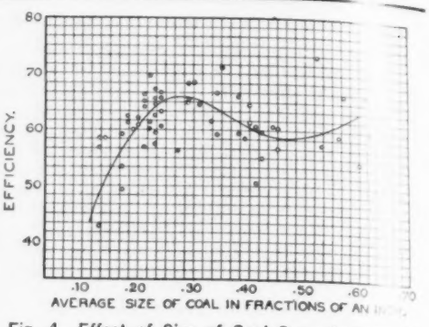


Fig. 4. Effect of Size of Coal Screenings on the Efficiency of a Boiler.

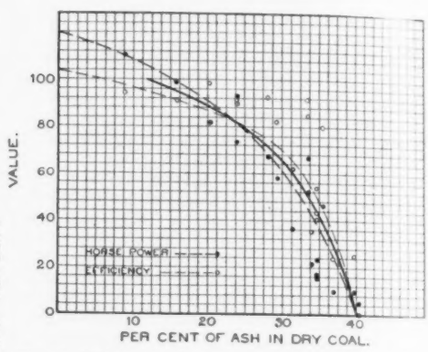


Fig. 5. Value of Coal as Regards Ash Content.

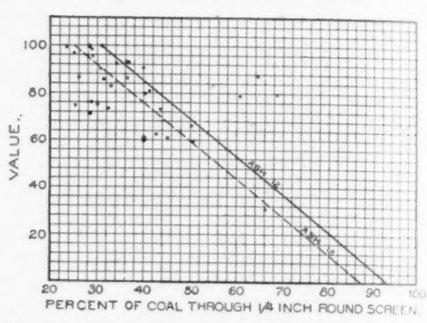


Fig. 6. Value of Coal as Regards Size; for Ash Content of 12% and 18%.

combustion and still further increasing the adulteration of the furnace gases with free air. When the temperature of the furnace has been thus reduced to about 600° F. the boiler is unable to absorb any more heat than is necessary to make up for radiation losses.

It will be observed that the points on the diagram do not fall in symmetrical order. This is particularly true of ash percentages of about 34, which may be explained by the refuse used in these tests being probably of a more fusible character than that used with the others. These tests with the ash composition were made with the smaller boiler above mentioned.

An excess of air accompanies a thin fire, and because of it, efficiency produced through the boiler is affected. On the other hand, a thick fire reduces the excess of air, but increases the volume of hydrocarbon gases which leave the surface, or, in other words, makes more smoke. If a furnace is located between the boiler and stoker, these gases will be burned; otherwise they will largely escape among the tubes of the boiler, as they did in this case. Therefore, under these conditions, a thin fire increases the loss due to excess of air, but decreases that due to smoke and incomplete combustion. On the other hand, a thick fire reduces the excess of air, but increases the smoke and escaping combustible gas, and so the best thickness of fire may be a matter of importance. With this type of boiler of a height of nine tubes, when served with chain grate stokers discharging immediately among the tubes, it is always most economical to produce as large a volume of smoke as possible with the coal being used. With boilers of 14 tubes in height the conditions are different, because such boilers are more efficient.

The curves of Figs. 3 and 4 illustrate the result of 62 tests. With each the size of coal was measured by screens with square openings ranging in dimensions from 0.25 to 1.50 ins., advancing by 0.25-in., and the average sizes of coal as shown at the base of the diagrams were calculated from sizing tests made with these screens, and

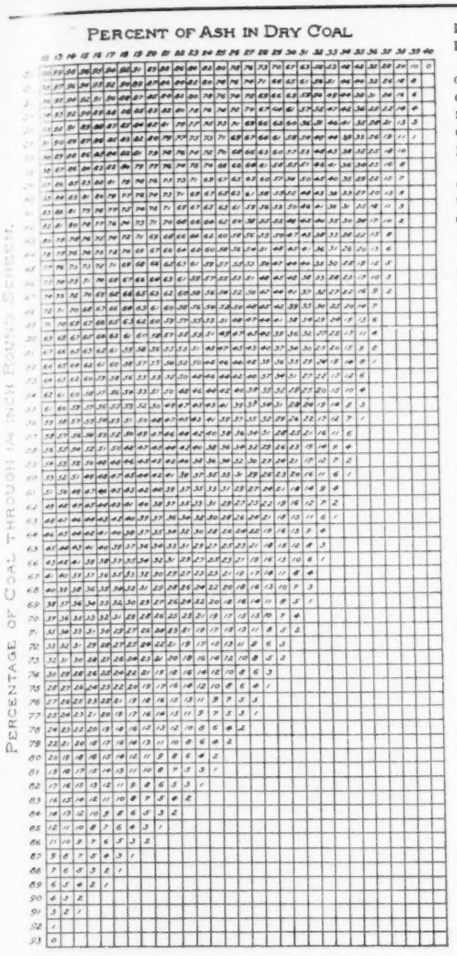


Fig. 7. Value of Screenings, as Compared with Pea Coal, for Different Percentages of Ash and Different Sizes of Coal.

represent the dimension in fractions of an inch of openings in a screen which would allow one-half of the coal to pass through and the other half to go over the screen, and it is this that is designated as its average size.

Fig. 3 shows the effect produced on horse-power output owing to this variation in size of the coal, and Fig. 4 illustrates the resulting efficiency from the same cause and for the same tests.

The curves for both efficiency and capacity drop midway between the tests with both small and large coal. This is a peculiarity which may be explained as follows: Performance becomes better as the size of coal increases, until a point is reached when the quantity of large pieces becomes so great that there is not enough fine material to properly close the interstices between, with the result that performance drops off, due to excess of air, until a condition is reached when all of the pieces of fuel approach uniformity, when, owing to greater agreement in size, they fit together better, and in a measure produce a homogeneous mass similar to that secured by the fine dust filling the spaces in the fuel bed in the first case.

The presence of fine dust in excess is a great and important source of trouble. Referring to Fig. 1, coal through an 0.25-in. square screen produced only 108 HP., yet a size of fuel known in Illinois as No. 5 washed coal, which will pass through a 0.25-in. round hole (a smaller aperture than the square opening), produced as high as 600 HP. under the same boiler.

The value of coal screenings is affected by four variables, which are: heating power, moisture, ash and size of the pieces of coal. The heating power of Illinois and Indiana pure coal—that is, coal free from ash and moisture—ranges from 13,800 as the minimum to a maximum of 14,500 B. T. U. per lb., and moisture from about 9% to 14%. These two characteristics, however, are of minimum importance, as either can affect the result by only a comparatively small amount. With the other two features, amount of ash and size of the pieces, each may exert an influence of such moment that they cause the fuel to be valueless. Thus, in fuel inspection service it may be necessary to test only the latter characteristic. Therefore the accompanying table, based on the variation in per cent. of ash and on the size of the coal, moisture and heating power being assumed as constant, has been prepared, showing the value of the screenings as compared with pure coal.

The fuel in service under a boiler produces two results, one of efficiency and the other of capacity, and capacity, or, in other words, quantity of steam produced, is a matter of great importance. Therefore, if these two effects may be averaged and treated as a single value, the problem is much simplified.

Inspection of Figs. 3 and 4 shows that the resulting capacity and efficiency vary approximately the same, and that the condition of fuel which results in a high efficiency also produces large capacity. This makes it possible to assign two values to the fuel, one applying to ash content, the other to its size, each of these values embracing efficiency and capacity as a unit. Therefore, in preparing the table for the value of screenings, a heavy curve was drawn midway between those of efficiency and capacity in Fig. 2. This average curve represents the value of the fuel as far as ash is concerned, which appears to be 100%, with 12% of ash in the dry screenings (Fig. 5); and according to this, value could be greater than 100%, but 12% represents an average minimum ash content for coal screenings in Illinois and Indiana at the present time; therefore such fuel is the best obtainable, and for this reason may have a value of 100% assigned to it.

The feature of size is a more difficult problem to solve than that of ash, and as an analysis made it appear that the presence of finer sizes was sometimes harmful and in other cases beneficial, the scheme of average size described in the first division of this paper was adopted in making these tests.

The average ash in the dry coal for these tests was 18%. A curve representing values for this ash content, and also for standard ash of 12%, were plotted (Fig. 6), the quantities of coal being transposed from that through the 0.25-in. square to a 0.25-in. round screen. The final result for ash shown by Fig. 5, and that for size by Fig. 6 furnished the data used for calculating the final combined values in the table for values of screenings, from which it appears that screenings having 12% of ash or less and of a size that no more than 31% will pass a 0.25-in. round screen may be considered of 100% value, or, in other words, sufficiently good for the purpose.

Basing conclusions on the measurement given by any one screen is arbitrary. Therefore, while the table of values may be depended on to identify all of the coal containing an excess amount of fine sizes, it will in a few cases condemn satisfactory fuel.

This paper does not presume to lay down the ultimate laws by which fine coals may be graded in value, but rather to point out the fact that such laws, although at present obscure, do exist, and that our conclusions drawn from numerous tests are as herein indicated.

NOTES ON THE WATER SUPPLY OF NEW YORK.

The average daily amount of water supplied to the Boroughs of Manhattan and the Bronx, New York City, during 1905 was 316,300,000 gals., according to an advance copy of the report of Mr. I. M. de Varona, Chief Engineer of the Department of Water Supply, Gas and Electricity. Of this quantity, 279,700,000 gals. came from the new Croton aqueduct, 18,500,000 from the old aqueduct, and 17,700,000 was drawn from the Bronx and Byran drainage areas. The average daily consumption of Croton water has increased gradually from 248,900,000 gals. in 1900 to 301,500,000 gals. in 1905. On the basis of a population of 2,390,000 for the Boroughs of Manhattan and the Bronx, the average daily per capita consumption in 1905 was 133 gals. It is estimated that the per capita consumption will rise to 143 gals. in 1910 and to 153 gals. in 1915. In the latter year, with an estimated population in the two boroughs of 3,262,000, the total average daily consumption would be 499,000,000 gals.

The rainfall and run-off in the Croton drainage area for the past six years have been as follows:

	Rainfall,	Run-off.	
	Ins.	Ins.	Per cent.
1900	48.19	23.6	49
1901	63.75	36.81	58
1902	53.48	33.79	63
1903	58.83	34.99	59
1904	50.70	25.23	50
1905	46.67	20.84	45

On the present basis of per capita increase of consumption, the total yield of the Croton drainage area when fully developed, supplemented by the yield of the Bronx (total, 345,000,000 gals. per day), would be sufficient for the two boroughs through the year 1907 only. This, it will be understood, is "on the basis of a minimum safe supply." Mr. de Varona suggests that it might be practicable, if legal restrictions were removed, to pump water from Dutchess County into the Croton drainage area and thus deliver a temporary supplementary supply to New York through the present aqueducts.

On the possibilities of additional water from Long Island, the following interesting remarks may be quoted from Mr. de Varona's report:

In regard to Suffolk County, it may be safely asserted that no other source of supply presents, under existing conditions, the same advantages as regards the early availability of the supply. From Suffolk County we can obtain a supply which may be safely estimated at about 180,000,000 gals. per day. An allowance of 60,000,000 gals. daily to Brooklyn would suffice for the needs of that borough beyond 1915, leaving the other 120,000,000 gals. for Manhattan and the Bronx. For the relief of Brooklyn 25,000,000 gals. per day could be readily obtained in about one year after the commencement of work at a cost of about $1,000,000, not including a new conduit or high-pressure engines, and, by the methods of construction adopted in Brooklyn, the estimated total supply of about 180,000,000 gals. per day could be developed within four years, assuming that all the water were to be delivered at the Ridgewood reservoirs.

Without much further investigation, close estimates cannot, obviously, be given, but for present purposes it may be assumed that the 60,000,000 gals. per day to be delivered to Brooklyn would cost, say, $10,000,000, and the 120,000,000 gals. for the supply of Manhattan and the Bronx would cost, say, $37,000,000. The disproportion in the cost between the respective quantities allotted to Brooklyn and Manhattan and the Bronx is thus explained:

(1) For the Brooklyn supply there would be no need to provide much of the land required for the conduits.

(2) Brooklyn would naturally take the water from the first section of Suffolk County developed and adjoining the Nassau County works.

(3) The cost of piping from Ridgewood to Central Park in Manhattan and tunneling under the East River, does not enter into the Brooklyn estimates.

The above figures represent in all cases the cost of supplying the amounts given, i. e., a total of 180,000,000 gals. a day. Good engineering practice would require designing the works for a larger probable yield, with a consequent increase in cost.

Mr. de Varona believes that the amount of water waste in New York has been exaggerated by many who have discussed the subject, and that it is "unsafe and unwarranted" to assume "the saving of waste exceeding 15% of the consumption." But, as he well points out, even this percentage, on the present basis of consumption, would amount to about 45,000,000 gals. a day, or approximately the annual normal increase in consumption for three years. He recommends the establishment of a separate division of his bureau, in charge of an assistant engineer experienced in such work, for the purpose of carrying out water waste investigations. After discussing and dismissing the Deacon water waste and the Venturi meters—the first as expensive in installation and maintenance and troublesome in repair, and the second as expensive to install and not readily removable subsequently—he speaks favorably of the pitometer as a device for use in making studies of waste in the distribution system.

Owing to the unpopularity of meters in New York, Mr. de Varona does not recommend the immediate adoption of the universal meter system, although he truly says, "That the metering of the supply is the true and permanent corrective of preventable waste, is the unanimous opinion of all the engineers in charge of our water-works throughout the country." He advises that, under existing conditions in New York, meters should be furnished and set at the cost of the city, instead of the householder as at present, and that a flat minimum rate should be made, in order not to discourage the free use of water in the tenement districts nor to give landlords an excuse for raising rents; and that the installation should be gradual, so as to distribute expense and give the benefit of information regarding the results to be obtained by the system. This method, he thinks, should be introduced first in several typical districts.

While believing that the Croton water supply should be filtered, Mr. de Varona is also of the opinion that a comprehensive joint sewerage system for villages in the Croton drainage area should be carried out jointly by the villages and by the City of New York. Some preliminary work on plans for such a system has been done, but it is highly desirable that there should be co-operation with the villages in this matter.

EQUIVALENT UNIFORM LIVE LOADS FOR RAILROAD BRIDGE TRUSSES.

By J. E. Kirkham,* Assoc. M. Am. Soc. C. E.

The object of this article is to present a convenient method for determining uniform loads equivalent to the typical wheel loads such as are now specified by the railroads in this country. The typical wheel loads referred to are loadings similar to those specified by Theodore Cooper and J. A. L. Waddell in their specifications for railroad bridges. The typical wheel loads specified by the different railroads while not alike in all respects, are quite similar, producing the same characteristic effects upon bridge trusses, and it is owing to this fact that it is possible to present a method that is general in its application.

The uniform load, in pounds per foot, equivalent to any typical wheel loads, for determining the stresses in the *web members* of any simple truss bridge, is

$$W = \frac{S}{\frac{1}{2}(L - d)} \qquad (A)$$

where

S = the maximum shear in the end panel, in pounds, due to the wheel loads.

L = length of span, in feet.

and d = panel length, in feet.

Then, by reducing the equivalent uniform load thus found by

$$\left(\frac{L}{100} + 2.5\right)\% \qquad (B)$$

we have the equivalent uniform load for determining the stresses in the *chord members*.

The shear in the end panel of any simple truss bridge, due to any typical wheel loads, can be determined very readily, even without the use of a moment table, and the substitutions in the above formulas are very simple, so that the equivalent uniform load for any simple truss bridge, of any length, can be determined in a very few minutes.

For example, let us determine the uniform load equivalent to Cooper's E40 loading for a

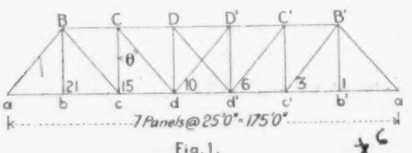


Fig. 1.

150-ft. single-track through bridge of 6 equal panels. The shear in the end panel due to the wheel loads is 162,000 lbs. Then the equivalent uniform load per foot of truss for determining the stresses in the web members is W = 162,000 ÷ ½ (150 — 25) = 2,592 lbs., and by reducing this by 4% = ($^{150}/_{100}$ + 2.5%) we get 2,488 lbs., which is the equivalent uniform load per foot of truss for determining the stresses in the chord members.

In order to show the manner of procedure in determining the stresses in truss bridges, we will compute the stresses in a 175-ft. single track bridge, shown in Fig. 1, due to the uniform load equivalent to Cooper's E40 loading.

The shear in the end panel due to Cooper's E40 loading is 189,900 lbs. Then the equivalent uniform load for determining the stresses in the web members is W = 189,900 ÷ ½ (175 — 25) = 2,532 lbs. per foot of truss and the panel load is P = 25 × 2,532 = 63,300 lbs. Then the stress

$$\text{in Ba} = 21 \times \frac{P \sec\theta}{7} = 247{,}000 \text{ lbs. compression;}$$

$$\text{in Bc} = 15 \times \frac{P \sec\theta}{7} = 176{,}300 \text{ lbs. tension;}$$

$$\text{in Cd} = 10 \times \frac{P \sec\theta}{7} = 117{,}600 \text{ lbs. tension or}$$

$$3 \times \frac{P \sec\theta}{7} = 35{,}300 \text{ lbs. compression;}$$

$$\text{in Dd'} = 6 \times \frac{P \sec\theta}{7} = 70{,}500 \text{ lbs. tension;}$$

$$\text{in Cc} = 10 \times \frac{P}{7} = 90{,}500 \text{ lbs. compression;}$$

$$\text{in Dd} = 6 \times \frac{P}{7} = 54{,}000 \text{ lbs. compression;}$$

in abs = 3 × P tan θ = 158,000 lbs. tension;
in BC and cd = 5 × P tan θ (100 — 4.25)% = 5 × P tan θ × 95.75% = 253,000 lbs.
in CD, DD' and dd' = 6 × P tan θ × 95.75% = 303,000 lbs.

The ordinary graphical method is the best to use in determining the stresses in bridges having curved chords—the equivalent uniform load making this possible.

It should be noted that the equivalent uniform load for determining the stresses in the bottom chord, abc, in the above truss, is the same as that used for determining the stresses in the web members, as the maximum stress in this member occurs when the shear in the end panel is a maximum. The same is true for all similar bridges.

In Figs. 2 to 6 a comparison is given between the stresses obtained by using wheel loads and those obtained by using equivalent uniform loads. The stresses marked W are computed from Cooper's E40 loading, while those marked U are computed from the equivalent uniform load.

Equivalent uniform loads for "higher structures," such as draw-spans, arches and cantilevers, can be determined in the same manner as given above provided judgment is used in applying the reduction formula (B).

If a system of wheel loads is used often, as would be the case in a railroad office, it is advisable to construct a curve similar to that shown in Fig. 7, which is for Cooper's E40 loading. The ordinates to this curve give the equivalent uniform loads for determining the stresses in the web members, and on a line below the curve are given the percentages by which the equivalent uniform load (the ordinate) is to be reduced to obtain the uniform load for determining the stresses in the chord members.

The two formulas (A) and (B) given above are empirical, being derived experimentally—at least proven that way—but at the same time they possess certain rational characteristics. The first step in the derivation of formula (A) was guided by the fact that any uniform load giving

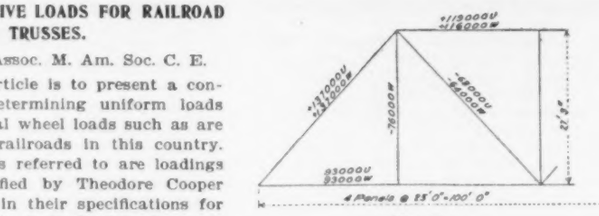


Fig. 2.

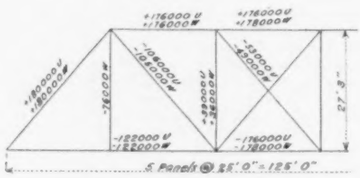


Fig. 3.

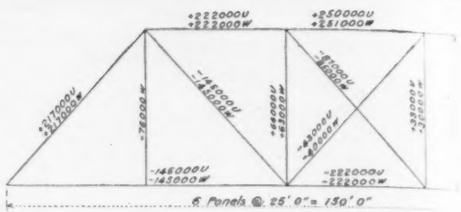


Fig. 4.

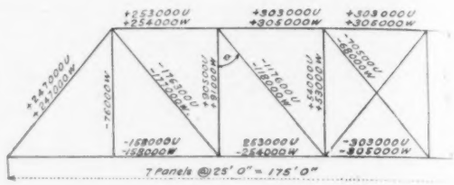


Fig. 5.

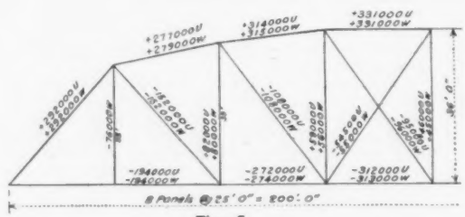


Fig. 6.

FIGS. 2 TO 6. COMPARISON OF STRESSES COMPUTED BY WHEEL-LOAD AND BY EQUIVALENT UNIFORM LOAD, FOR FIVE DIFFERENT TRUSSES, FROM 100 FT. TO 200 FT. IN SPAN.

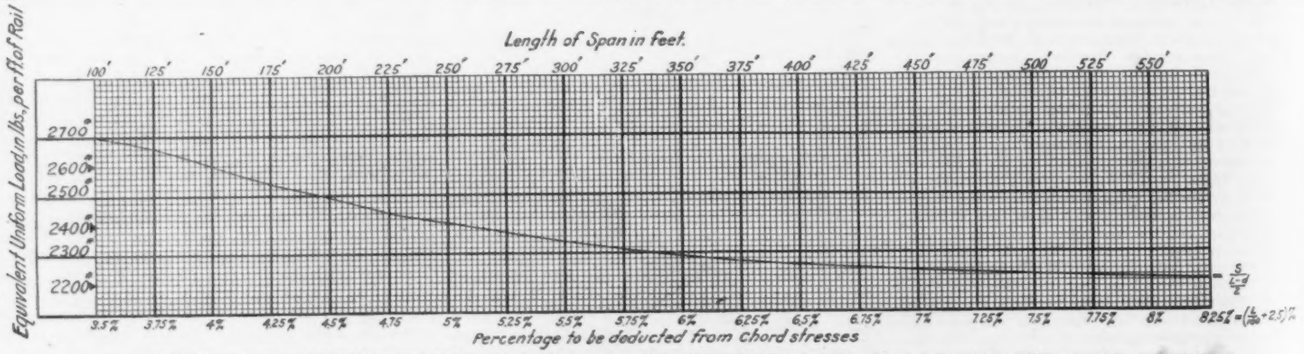


FIG. 7. DIAGRAM GIVING EQUIVALENT UNIFORM LOAD, AND REDUCTION-PER CENT. FOR LOADING FOR CHORD STRESSES; FOR COOPER'S E40 LOADING.

*Assistant Engineer, American Bridge Co., Ambridge, Pa.

the same stresses in the end panel of a truss bridge as a system of wheel loads, must produce the same shear in that panel. We know that the maximum shear in the end panel due to a uniform load occurs when this load extends over the entire length of the span, and is equal to ½ W (L — d), where the letters signify the same as stated above. This, then, must equal S, the shear produced by the wheel loads. Therefore, ½ W (L — d) = S, or,

$$W = \frac{S}{\frac{1}{2}(L - d)},$$

which is formula (A). It is obvious that the uniform load W thus determined will give the correct stresses in the end panel of a truss bridge. So we start with no false assumption. It is then found by experiment that the equivalent uniform load W which gives the correct stresses in the end panel will also give approximately correct stresses for the web members throughout the span; but will give excess values for the chord members. Thus the necessity of the reduction formula (B) is evident.

The reduction formula (B) was derived in the following way: The stresses due to a system of typical wheel loads and those due to the equivalent uniform load, determined by formula (A), were computed in truss bridges ranging in length from 100 ft. to 500 ft., the panel lengths varying in the different spans so as to give economic designs. The excesses of the stresses in the chords at the quarter point produced by the uniform load over those produced by the wheel loads were found in this way and the formula (B) was then determined by experiment so as to give this excess in percentage of the uniform load found by formula (A).

REPRESENTATION OF WELLS AND SPRINGS ON MAPS.*

By Myron L. Fuller.†

In general there has been no attempt at uniformity of practice in the delineation on maps of underground water features or of wells or springs, but the increase in the number of men engaged in underground water investigations, both on local and national surveys, and the increasing number of reports issued has been so rapid within the past two or three years that it now appears desirable that a concerted movement be made to develop a uniform system of symbols for use on maps. As by far the greater portion of the underground water literature, other than that in engineering magazines, is published by the U. S. Geological Survey, it is thought that the adoption of some such system by its members will go far toward securing uniformity in the country as a whole.

It is believed that the various types of wells and springs can be best shown by symbols of different colors, but unfortunately colored maps cannot always be had, and it becomes necessary to represent a considerable number of features in black and white. Symbols which can be readily and quickly made are also a great convenience in note keeping in the field and result in much saving of time.

The number of symbols devised should be sufficient for the representation of all features which it is desirable to show. If wholly arbitrary devices are used, confusion will result whenever a considerable number are used simultaneously, but this difficulty will be largely avoided if the system adopted is based on a few suggestive forms grouped according to easily remembered principles. Unnecessary duplication should, of course, be avoided, although even a multiplicity of symbols leads to less confusion than the attempt to use a single device to represent several different things.

The older generalized maps, such as the early topographic sheets of this country, have given place to maps in which the features are shown in great detail, yet not only are the maps not crowded and confusing, but their usefulness has been increased many fold. It is believed that the use of any necessary number of symbols will likewise add to the usefulness of underground water maps. The system should, in fact, be as nearly complete as is practicable, for although the use of many symbols on a single map might be objectionable, in reality only a few would, in most cases, be used at one time.

The principles to be considered in devising a system of well and spring symbols for underground water maps are (1) simplicity, (2) clearness, (3) ease of making, and (4) suggestiveness. Failure to answer these various requirements ruled out many of the arbitrary systems used in the past, although several of the old symbols have been utilized in the new system proposed.

It is believed that a system of symbols can be most logically developed if a single arbitrary device is taken as a base. In common practice a circle is most often used for a well, while more or less closely allied devices are used for springs. Inasmuch as both wells and springs are ordinarily approximately circular, this device, which seems to have both the required simplicity and suggestiveness, is proposed.

WELL SYMBOLS.

BASE SYMBOL.—The cross section of all but a few shallow open wells being circular, the unmodified circle is proposed as a base symbol to indicate all wells.

SUCCESSFUL AND UNSUCCESSFUL WELLS.—The most significant feature of a well from the economic and practical standpoint is its success or failure, and provision for its representation has to be made at the very start. It is thought that a successful well—a "full" well in many instances—can be best shown by a filled circle, or rather a circular dot, while an empty or unsuccessful well can be best shown by a simple circle.

NON-MINERAL AND MINERAL WELLS.—Next to obtaining water its quality is most important, and it becomes necessary to devise a symbol for indicating the mineral property. If an ordinary non-mineral well is represented by a circle, a mineral well, or one in which the water includes mineral matter in solution, may well be represented by a circle inclosing a dot.

HYDROSTATIC PRESSURE IN WELLS.—Next to quantity and quality the problem of whether or not the waters will rise is of the greatest importance, and a further symbol for distinguishing the wells simply sunk to the water table and in which the water does not rise from those sunk to confined artesian waters becomes necessary. It is thought that a vertical line can be best used to designate the vertical rise of the water. Such a line, to be superimposed on any of the other well symbols, is therefore recommended.

FLOWING WELLS.—The wells in which the waters rise are still further subdivided into those which fail to reach the surface and those which flow. For the latter the plus sign has often been used and is proposed in the present instance. It is to be superimposed on other well symbols as in the case of the vertical line.

WELLS FROM DIFFERENT HORIZONS.—In many areas all of the successful wells do not draw from the same horizon, in some instances as many as four or five different water-bearing beds being utilized. As no limit can be put to the number of horizons which it may be necessary to indicate, and as the horizons do not necessarily have any relation to the character of the well, it has not seemed desirable to devise symbols for their representation. Instead it is recommended that the horizon of the supply be indicated by letters placed to the left of the well symbol, the space to the right being left for the insertion of figures giving the depth, height of water, elevation, etc.

SPRING SYMBOLS.

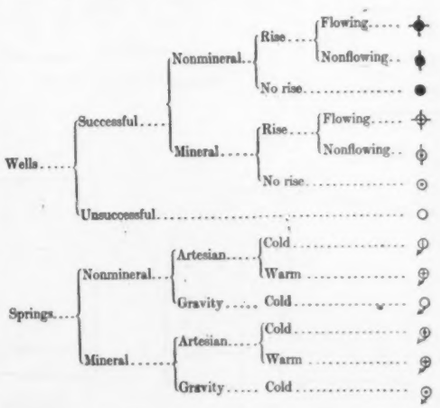
BASE SYMBOL.—As a base symbol for springs a circle with a short irregular line, indicative of a stream, leading away from the circumference, is considered as most in harmony with common usage, especially in topographic maps. All springs yield water, so there is no demand for distinguishing springs as in the case of successful and unsuccessful wells.

MINERAL AND NON-MINERAL SPRINGS.—The presence of mineral matter in the water may be indicated by a dot placed in the center of the circle as in the case of the wells.

SUPERFICIAL AND ARTESIAN SPRINGS.—In some springs the water is unconfined, the flows taking place where the surface of the water table is cut by a depression. The movement of the water in such instances is almost entirely downward, and the springs are frequently spoken of as superficial or gravity springs, since the water has not been to any distance below the water table and emerges under the direct action of gravity. The water of such springs will not rise if confined. In artesian springs, on the other hand, the water comes from below and rises under the influence of hydrostatic pressure, and when confined will sometimes rise to considerable heights above the spring mouth. In such instances it is recommended that, as in the case of the wells, the hydrostatic principle be represented by a vertical line superimposed on the base symbol, but not extending beyond the circumference of the circle. Flows do not need to be represented, since all springs possess this property.

THERMAL PROPERTY.—Springs may be further divided into cold or warm, but only one symbol, that for the thermal property, is required. For this a horizontal bar is proposed as the simplest device which can be superimposed on all others. It is intended that this bar shall not extend beyond the circumference of the circle so that there may be no confusion with the device indicating flowing wells.

APPLICATION.—The use of the various devices is illustrated herewith. The arrangement is not intended as a classification of wells and springs, but is simply for convenience in showing the use of the symbols.



Symbols for Wells and Springs Suggested by the Division of Hydrology, U. S. Geological Survey.

THE PRODUCTION OF SAND-LIME BRICK in the United States during 1905 is valued at $972,064, as compared with $463,128 in 1904 and $155,040 in 1903. The number of plants engaged in manufacturing these bricks was 84, 57 and 16, respectively, for the years named. The State of Michigan led in 1905, with 12 plants and a total product valued at $169,302. New York followed, with 7 plants and an output value of $123,104. Other details of the production of sand-lime brick may be found in an advance chapter from "Mineral Resources of the United States, Calendar Year 1905," entitled "The Production of Lime and Sand-Lime Brick in 1905."

STREAM POLLUTION BY TRADE WASTES is being investigated jointly in Ohio by the Hydro-Economic Division of the U. S. Geological Survey and the Ohio State Board of Health. The "Ohio Sanitary Bulletin" for April-June, 1906, is devoted almost wholly to a report on "Disposal of Waste Water from Cloth Dyeing and Finishing Works at Ravenna, Ohio," by Mr. Herman Stabler, Assistant Engineer, U. S. Geological Survey. The plant in question is owned by the Cleveland Worsted Mills Co. The analytical studies were made under the direction of Dr. E. G. Horton, Bacteriologist and Chemist to the Ohio State Board of Health. Estimates indicated that with "the town fully sewered, the domestic sewage would be in quantity only twice the flow from the factory." Responsibility for the disposal of the factory waste has been assumed by the local authorities; the problem therefore was to decide whether it was best "to treat the waste alone or in connection with the town sewage and what processes of purification would give the best results at a reasonable cost." The paper (which doubtless can be secured by sending postage stamp and address to the State Board of Health, Columbus, Ohio) describes the operations carried on at the plant in question, and sets forth in detail the character of the waste waters. The paper also gives the results of various experiments in the treatment of the wastes. Mr. Stabler's summary of his conclusions and recommendations is as follows:

(1) At Ravenna, Ohio, waste waters from a dyeing and finishing establishment pollute a small stream and thereby have caused complaints and the institution of suits by lower riparian owners.

(2) The processes giving rise to the waste waters are the washing (not scouring) and dyeing of cloth and wool. The liquors are highly colored, soapy, and have a rather disagreeable odor. They are not putrescible when undiluted but cause putrefaction in the stream into which they are discharged. They carry shreds of cloth and wool, have a high temperature, bear a considerable amount of organic matter in solution, and are highly charged with chemicals among which sodium sulphate (Glauber's salt) is used in greatest quantity.

(3) Experiments at Ravenna and elsewhere have shown that partial purification of various degrees can be obtained by straining through coke, by coke contact filtration, by intermittent continuous filtration through broken stone, and by chemical precipitation. Filtration through sand and chemical precipitation followed by filtration give a high degree of purification. All of the foregoing methods will prove too costly for a separate treatment of factory wastes in the Ravenna problem. Recovery of valuable materials from the waste waters is of doubtful economy as an adjunct to purification and is not desirable at Ravenna. Septic treatment with sewage followed by bacterial filtration through sprinkling filters has been attended with success on somewhat similar liquors in England.

(4) Experimentation along the lines of oxidation with subsequent sedimentation or filtration is believed to be the best field for future attempts at disposal.

(5) With present knowledge upon the subject, the best results at Ravenna can probably be obtained by disposal in an elastic system of sedimentation or chemical precipitation followed by septic treatment, filtration, and sedimentation, together with the domestic sewage of the town, special provisions for proper operation and maintenance being provided.

*Slightly condensed from Water-Supply and Irrigation Paper No. 160, U. S. Geological Survey.

†Geologist in charge of Underground Water Supplies, U. S. Geological Survey, Washington, D. C.

HIGH-SPEED TOOLS FOR RAPID WORK IN TURNING LOCOMOTIVE DRIVING-WHEEL TIRES.

With the use of high-speed steel for tools used in turning driving-wheel tires great reductions have been made in the time required for this work. At many shops equipped with high-speed tools and lathes of sufficient strength and power to enable these tools to be worked to their full advantage, from six to ten pairs of wheels are now turned in a ten-hour day. Fig. 1 shows the form of some of the special tools used in work of this kind. The roughing tool is first used, roughing out the tread in ten or eleven revolutions, and the top and both sides of flange in about five revolutions. The scraper is then used to smooth up the tread; this usually requires two revolutions, but may sometimes be done in one revolution if the roughing cut has given approximately the correct size. The inside and outside of the flange are then finished with the scrapers or forming tools, about two revolutions being required to finish each side of the flange. The outside of the tread is then finished off with the chamfering or beveling tool, and the wheels are then ready to be removed. For gouging out hard spot a special tool is used; this is shown in the illustration. The tools are of air-hardening steel and have a cross section in the bar of at least $1\frac{1}{2} \times 3$ ins., as smaller sizes are not strong enough to take the heavy cuts and feeds of a modern lathe designed for high-speed work.

In tests of a lathe of this kind at the New York Central Ry. shops at West Albany, N. Y., ten pairs of wheels were turned in an average of 54.6 minutes per pair, while 15 pairs were turned in an average of 53.6 minutes per pair. In the latter test, most of the cutting was done at a speed of 12¾ ft. per min., and the chips turned off weighed about 2,860 lbs. Where no hard spots were encountered the wheels were turned complete in from 21 to 23 revolutions as follows: Eight or nine revolutions for the roughing cut on the tread; three across the top of the flange; and one on each side to rough down the front and back of the flange. The scraper was then applied to the tread of the wheel, smoothing it up in two revolutions. Two revolutions were required for cutting the outer bevel and chamfering the outside corners of the tread, and two on each side for finishing the flange. The conditions and results of the two sets of tests are summarized in the following table:

	10 Tires.	15 Tires.
Diameter of wheels	64 and 79 ins.	50 to 72 ins.
Cutting speed, ft. per min.	8½ to 14	12¾ to 14
Feed per revolution	15/32 in.	15/32 in.
Depth of cut	5/8 to 9/16 in.	¾ to 9/16 in.
Putting in wheels		6 to 8 mins.
Cutting	42 to 52 mins.	39 to 52 mins.
Taking out wheels		2 to 4 mins.
Changing wheels	8 to 10 mins.	
Total time	546 mins.	804 mins.
Average time per pair	54.6 mins.	53.6 mins.

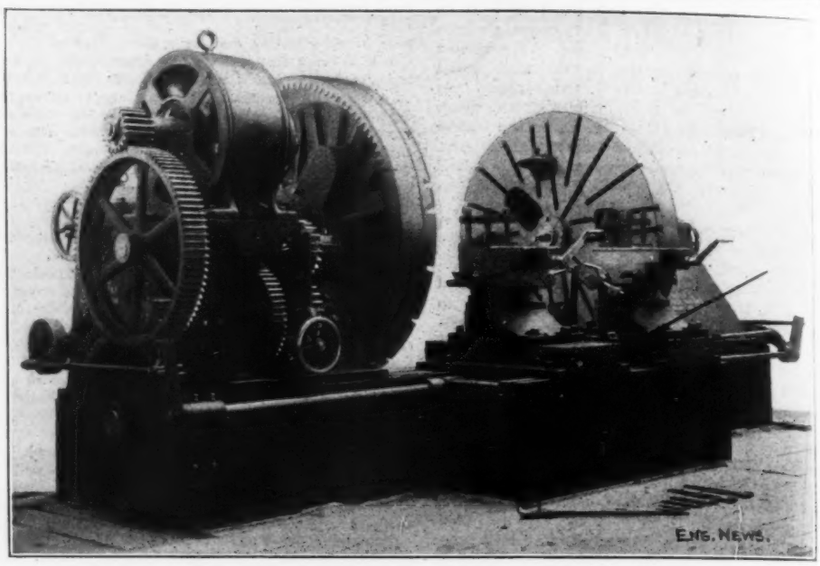


FIG. 2. 90-IN. DRIVING-WHEEL LATHE FOR HIGH SPEED WORK.
Niles-Bement-Pond Co., New York, Builders.

This work was done on a 90-in. lathe, built by the Niles-Bement-Pond Co., of New York, N. Y. A view of this is shown in Fig. 2, but, as erected in the shops the top of the bed-plate is level with the floor. The machine is designed to take two cuts ½-in. deep with 3-16-in. feed at a speed of 20 ft. per min.; in service, however, it has done heavier work than this. The face-plates are 90 ins. in diameter and normally 6⅞ ft. apart, but one of them can be traversed on the bed to give a maximum clearance of 9 ft. in order to admit a pair of main driving wheels with their crank pins. Holes in the face-plates admit these pins, so that the wheels can be fastened close to the plates; the fastenings hold the wheels by the rims instead of by the spokes, thus removing the tendency to vibration or chattering under heavy feeds and cuts. The swing over the bed is 92 ins., and the machine will take in wheels of 50 to 84 ins. diameter without changing the positions of the carriages. The machine is driven by gearing from a 40-HP. Westinghouse motor having a speed variation of 1 to 2; this, in combination with changes by gearing, provides cutting speeds of 10 to 25 ft. per min. on all diameters from 48 to 84 ins. It will be seen that the face-plates are driven by internal gears. A 5-HP. motor is used for traversing the moving head and face-plate. During tests the driving motor has had to develop as high as 60 HP. at times.

EXPERIMENTS ON THE STRENGTH OF TREATED timber are reported by W. Kendrick Hatt, Assoc. M. Am. Soc. C. E., Civil Engineer U. S. Forest Service, in Circular 39, Forest Service, U. S. Department of Agriculture. The conclusions drawn by Professor Hatt are as follows:

(1) A high degree of steaming is injurious to wood. The degree of steaming at which pronounced harm results will depend upon the quality of the wood and its degree of seasoning, and upon the pressure (temperature) of steam and the duration of its application. For loblolly pine the limit of safety is certainly 30 lbs. for 4 hours, or 20 lbs. for 6 hours.

(2) The presence of zinc chlorid will not weaken wood under static loading, although the indications are that the wood becomes brittle under impact.

(3) The presence of creosote will not weaken wood of itself. Since apparently it is present only in the openings of the cells, and does not get into the cell walls, its action can only be to retard the seasoning of the wood.

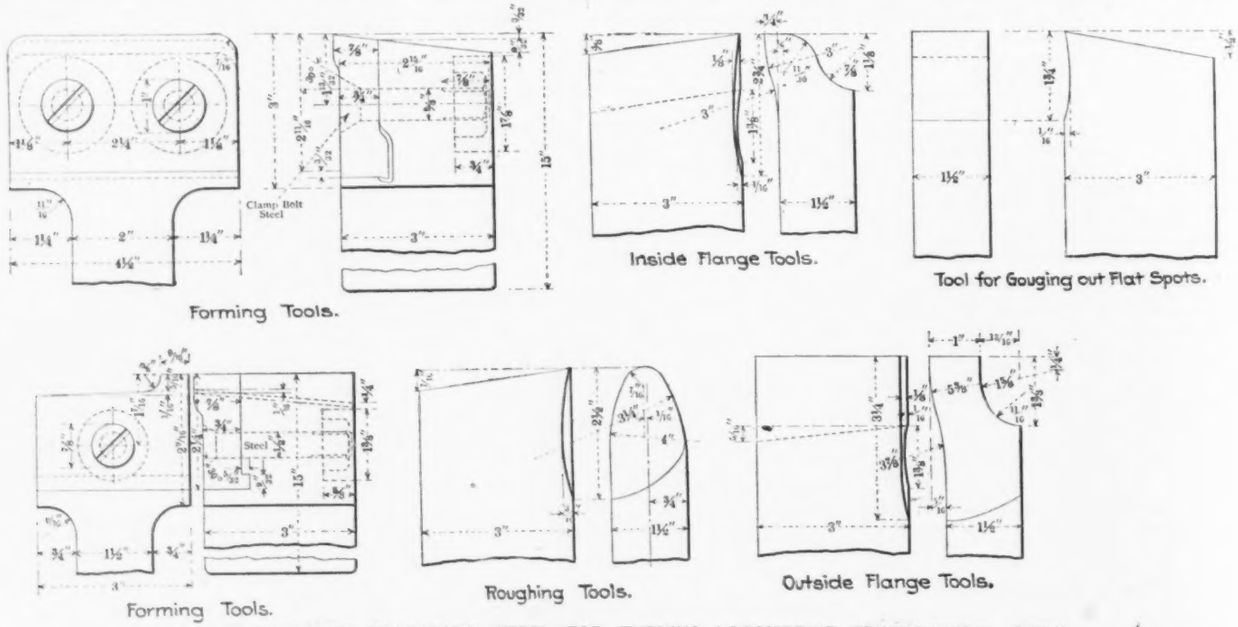
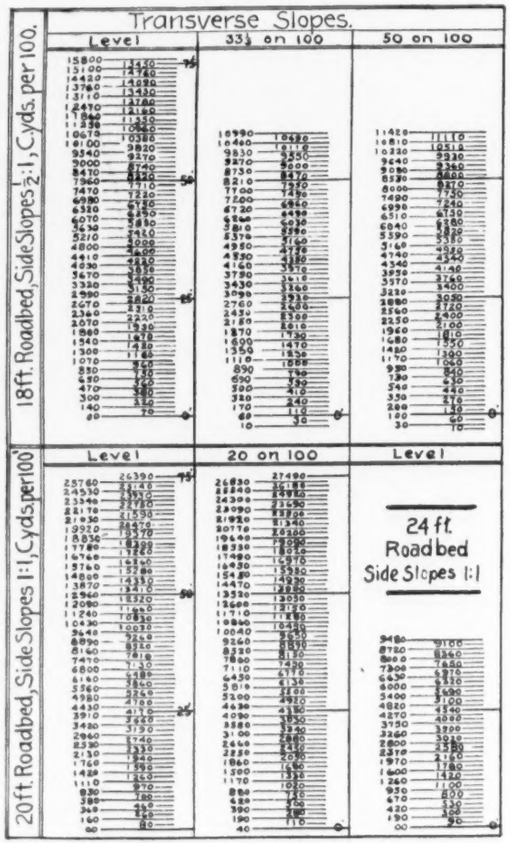


FIG. 1. TOOLS OF HIGH-SPEED STEEL FOR TURNING LOCOMOTIVE DRIVING-WHEEL TIRES.

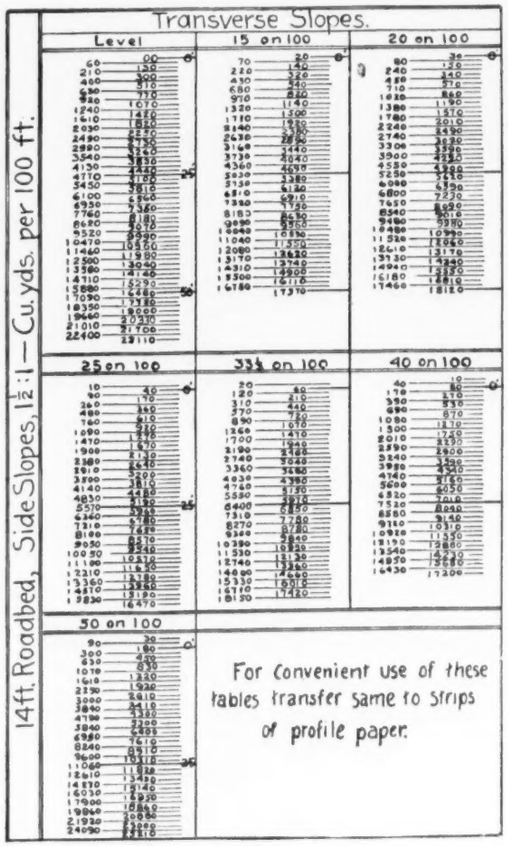
I.

Excavation Tables.



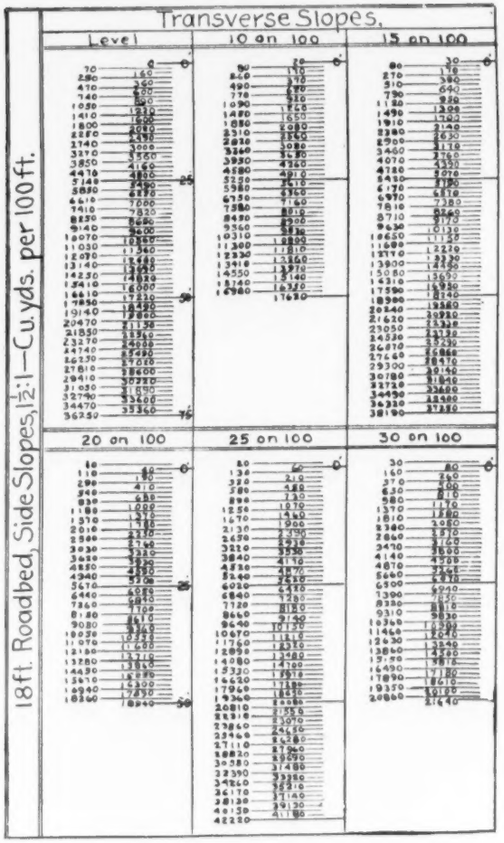
II.

Embankment Tables.


Transverse Slopes.
14ft. Roadbed, Side Slopes, 1½:1—Cu. yds. per 100 ft.
Level
15 on 100
20 on 100
25 on 100
33⅓ on 100
40 on 100
50 on 100
For convenient use of these tables transfer same to strips of profile paper.

III.

Embankment Tables.

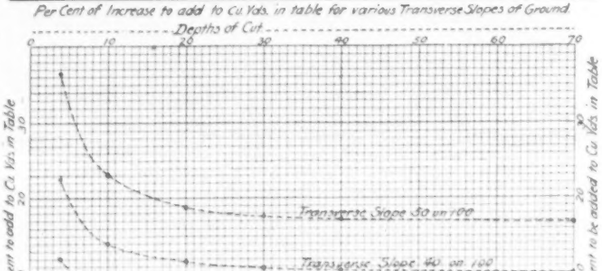


IV.

Table of Level Cuttings—Cu. Yds. per 100 ft.—18 ft. Roadbed. Side Slopes ¾ to 1 (or ¾:1)

D	Cu. Yds	D	Cu. Yds	D	Cu. Yds	D	Cu. Yds	D	Cu. Yds	D	Cu. Yds	D	Cu. Yds	D	Cu. Yds	D	Cu. Yds
1	69	9	825	17	1936	25	3403	33	5225	41	7403	49	9936	57	12825	65	16083
2	144	10	944	18	2100	26	3611	34	5478	42	7700	50	10278	58	13211	66	16500
3	225	11	1069	19	2269	27	3825	35	5736	43	8003	51	10625	59	13603	67	16936
4	311	12	1200	20	2444	28	4044	36	6000	44	8311	52	10978	60	14000	68	17378
5	403	13	1336	21	2625	29	4269	37	6269	45	8625	53	11336	61	14403	69	17825
6	500	14	1478	22	2811	30	4500	38	6544	46	8944	54	11700	62	14811	70	18278
7	603	15	1625	23	3003	31	4736	39	6825	47	9269	55	12069	63	15225		
8	711	16	1778	24	3200	32	4978	40	7111	48	9600	56	12444	64	15694		

Per Cent of Increase to add to Cu Yds. in table for various Transverse Slopes of Ground.


Depths of Cut
Transverse Slope 50 on 100
Transverse Slope 40 on 100
Per Cent to add to Cu Yds. in Table
Per Cent to be added to Cu Yds. in Table

VI.

Single Track Box Culverts, 14 ft. Roadway.

Total Height Bank	2'x3' C.Y. Mas.	C.Y. Par.	2½'x3½' 3'x3' C.Y. Mas.	C.Y. Par.	3'x4' C.Y. Mas.	C.Y. Par.	3'x5' 4'x4' C.Y. Mas.	C.Y. Par.	4'x5' C.Y. Mas.	C.Y. Par.	Double 3'x4' C.Y. Mas.	C.Y. Par.
5	17	6	20	6	26	8	31	8	31	9	39	12
6	19	7	22	7	29	9	35	9	35	10	44	14
7	21	7	25	8	32	10	39	10	39	11	48	15
8	23	8	27	8	35	11	43	11	43	12	53	16
9	25	9	30	9	38	12	47	12	47	13	58	18
10	27	10	32	10	41	13	51	13	51	14	63	20
11	29	11	34	11	44	14	55	14	55	15	68	21
12	31	12	37	12	47	15	59	15	59	16	73	23
13	33	13	39	13	50	16	63	16	63	17	78	24
14	35	14	42	14	53	17	67	17	67	18	83	26
15	37	15	44	15	56	18	71	18	71	19	88	27
16	39	16	46	16	59	19	75	19	75	20	93	29
17	41	17	49	16	62	20	79	20	79	21	98	30
18	43	18	51	17	65	21	83	21	83	22	103	32
19	45	19	53	18	68	22	87	22	87	23	108	33
20	47	20	56	19	71	23	91	23	91	24	113	35
21	49	21	58	20	74	24	95	24	95	25	118	36
22	51	22	61	21	77	25	99	25	99	26	123	38
23	53	23	63	22	80	26	103	26	103	27	128	39
24	55	24	65	23	83	27	107	27	107	28	133	41
25	57	25	68	24	86	28	111	28	111	29	138	42
26	59	26	70	25	89	29	115	29	115	30	143	44
27	61	27	73	26	92	30	119	30	119	31	148	45
28	63	28	75	27	95	31	123	31	123	32	153	47
29	65	29	77	28	98	32	127	32	127	33	158	48
30	67	30	80	29	101	33	131	33	131	34	163	50
31	69	31	82	30	104	34	135	34	135	35	168	51
32	71	32	85	31	107	35	139	35	139	36	173	53
33	73	33	87	32	110	36	143	36	143	37	178	54
34	75	34	89	33	113	37	147	37	147	38	183	56
35	77	35	92	34	116	38	151	38	151	39	188	57
36	79	36	94	35	119	39	155	39	155	40	193	59
37	81	37	97	36	122	40	159	40	159	41	198	60
38	83	38	99	37	125	41	163	41	163	42	203	62
39	85	39	101	38	128	42	167	42	167	43	208	63
40	87	40	104	39	131	43	171	43	171	44	213	65
41	89	41	106	40	134	44	175	44	175	45	218	66
42	91	42	109	41	137	45	179	45	179	46	223	68

Box Culverts

Add for each additional foot in height for 2'x3', 2½'x3½' and 3'x3' Culverts, 2 cu. yds. Mas. 1½ cu. yds. Par.
" " " " " 3'x4' " 3 " " 2 " "
" " " " " 3½'x4', 3'x5' and 4'x4' " 4 " " 2½ " "
" " " " " 4'x5' " 5 " " 3 " "
" " " " " Double 3'x4' " 6 " " 4 " "

X.

TABLE OF MINIMUM CROSS-SECTION AREAS FOR WATERWAYS OF

VII.
A.

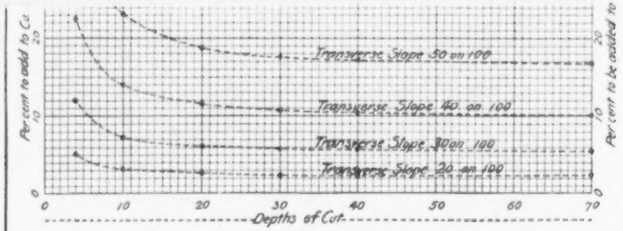
Plate Girders Deck Span Ft	Plate Girders Deck Weight Lbs.	Through Truss Span Ft	Through Truss Weight Lbs.
25	13000	100	130000
30	16000	125	175000
40	23000	150	220000
50	33000	175	280000
60	45000	200	350000
70	60000	250	540000
80	75000		

Through Girders weigh about 66 lbs more Steel Viaducts—Multiply profile area in sq. ft by 19; and when under 40 ft high by 20 or 21 to get lbs. Metal.

VII.
B.

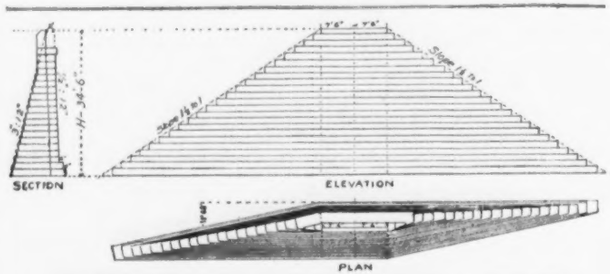
Through Truss Span Ft	Through Truss Weight Lbs.
100	140000
125	185000
150	260000

Steel Viaducts—(Coopers E 40 Loading.) Multiply profile area in sq. ft by 26, at 20' high, by 29 at 60 high, and by 17 at 90 high. For Cooper's E 50 loading about 20% heavier. See ad. page.


Per cent to add to Cu
Per cent to be added to
Transverse Slope 50 on 100
Transverse Slope 40 on 100
Transverse Slope 30 on 100
Transverse Slope 20 on 100
Depths of Cut

V.

TABLE OF CUBIC YARDS IN RETAINING WALLS AND ABUTMENTS FOR PLATE GIRDER BRIDGES



Height of Abut in Ft.-H	Square Crossing Straight Wing Walls Cu Yards in two Abuts.	Square Crossing Alternate Wings Flared Cu Yds. in two Abuts.	Skew Crossing Angle 30° Cu Yds. in two Abuts.
6	27	27	30
7½	41	45	45
9	57	59	63
10½	75	77	84
12	96	100	107
13½	121	125	134
15	149	155	164
16½	181	188	193
18	217	226	238
19½	257	265	282
21	302	316	331
22½	351	368	385
24	406	426	444
25½	466	488	508
27	531	557	578
28½	602	631	654
30	677	713	738
31½	760	800	827
33	848	894	923
34½	944	993	1026

These Tables make no allowance for offsets at the Toe, or Cut-offs at the end.

X.

TABLE OF MINIMUM CROSS-SECTION AREAS FOR WATERWAYS OF CULVERTS AND SMALL BRIDGES

Run-off Calculated by Burkli Ziegler and McMath Formulae; Maximum Rainfall 3 in per Hour; Coefficient, C = 0·3. Burkli-Ziegler Formula used only where S÷A exceeds unity. Burkli-Ziegler Formula, $Q = RC\sqrt{\frac{S}{A}}$ (Trautwine, p 575, Ed 1902); McMath Formula, $Q = RC\sqrt[3]{\frac{S}{A}}$. Assumed Maximum Velocity for culv running full = 6 ft per Sec.; Area of Waterway = $\frac{QA}{6}$. Maximum Discharge in cu ft per sec. = 6 × Opening in sq ft.

Area Drained	Mountainous		Hilly		Rolling		Flat			
	Slope Ft per 1000	Opening Sq Ft	Slope Ft per 1000	Opening Sq Ft	Slope Ft per 1000	Opening Sq Ft	Slope Ft per 1000	Opening Sq Ft	Slope Ft per 1000	Opening Sq Ft
Acres										
25	700	1·3	350	1·3	180	1·1	90	0·9	45	0·8
5	640	2·5	320	2·1	160	1·8	80	1·5	40	1·3
10	580	4·1	290	3·3	140	2·9	70	2·4	35	2·1
15	540	5·5	270	4·6	130	3·8	70	3·3	35	2·8
20	510	6·7	250	5·6	130	4·8	60	3·9	30	3·3
30	480	9·	240	7·6	120	6·4	60	5·4	30	4·5
40	460	11·	230	9·3	110	7·7	60	6·6	30	5·7
50	440	12·8	220	10·9	110	9·1	50	7·3	25	6·5
60	420	14·6	210	12·3	110	10·5	50	8·7	25	7·6
70	410	16·3	200	13·7	100	11·5	50	9·8	25	8·5
80	400	17·8	200	15·1	100	12·7	50	10·9	25	9·5
100	380	20·9	190	17·6	100	15·	50	13·1	25	11·4
120	360	23·7	180	19·9	90	17·	45	14·8	25	13·2
140	350	26·4	180	22·4	90	19·2	45	16·7	20	14·2
160	340	29·	170	24·4	90	21·4	45	18·6	20	15·8
180	330	31·4	170	26·7	80	23·	40	20·	20	17·4
200	320	33·7	160	28·7	80	25·	40	21·7	20	18·9
240	310	38·4	150	32·8	80	28·9	40	25·2	19	21·7
280	300	42·7	150	37·1	70	31·8	35	27·7	19	24·5
320	290	47·2	140	40·7	70	35·4	35	30·8	18	27·
360	280	51·4	140	44·7	70	38·9	35	33·9	18	29·7
400	270	55·3	140	48·6	70	42·4	35	36·9	17	31·9
450	260	60·3	130	52·6	70	46·5	35	40·5	17	35·1
500	260	65·8	130	57·3	60	49·1	30	42·7	16	37·7
550	250	70·5	120	60·8	60	53·	30	46·1	16	40·7
600	240	74·9	120	65·2	60	56·8	30	49·4	15	43·
Sq. Mi.										
1·	240	78·	120	69·	60	60·	30	52·	15	45·
1·25	230	84·	110	81·	60	71·	30	62·	14	53·
1·5	210	106·	110	93·	50	80·	25	69·	13	61·
2·	200	132·	100	115·	50	100·	25	87·	12	75·
2·5	180	155·	90	135·	45	118·	23	104·	11	89·
3·	170	177·	90	156·	45	136·	20	116·	11	103·
3·5	160	198·	80	173·	40	150·	20	131·	10	114·
4·	150	218·	80	192·	40	167·	19	144·	10	127·
5·	140	257·	70	224·	35	195·	17	168·	9·	148·
6·	130	293·	60	251·	30	218·	16	193·	8·	168·
7·	120	330·	60	286·	30	245·	15	215·	7·	186·
8·	120	363·	60	314·	30	275·	14	236·	7·	205·
10·	100	418·	50	364·	25	317·	13	278·	7·	246·
12·	100	484·	50	421·	25	367·	12·	316·	6·	275·
16·	90	596·	45	519·	20	441·	11·	392·	5·	334·
20·	80	686·	40	606·	19·	522·	10·	459·	5·	400·
25·	70·	810·	35·	705·	17·	610·	9·	537·	4·	457·
30·	60·	909·	30·	791·	16·	698·	8·	607·	4·	528·
35·	60·	1028·	30·	895·	14·	765·	7·	669·	4·	598·
40·	50·	1103·	25·	960·	14·	855·	7·	744·	3·	628·
50·	50·	1318·	25·	1148·	12·	991·	6·	863·	3·	751·
60·	50·	1524·	25·	1328·	12·	1147·	6·	998·	3·	869·
70·	45·	1690·	25·	1502·	12·	1297·	6·	1129·	3·	983·
85·	45·	1973·	20·	1678·	11·	1489·	5·	1272·	3·	1148·
100	40·	2195·	20·	1910·	10·	1664·	5·	1449·	3·	1308·

Use only as a Check-Check with H.W. Marks →

Not Applicable to paved streets or impervious city areas. When double pipes or double culverts are used add about 20% to total Cross-section Areas.

Through Truss		Steel Viaducts—(Coopers E 40 Loading)
Span Ft	Weight Lbs.	Multiply profile area in sq ft by 26 at 20' high; by 20 at 60' high, and by 17 at 90' high. For Coopers E 50 loading about 20% might be added.
100	140 000	
125	195 000	
150	260 000	
175	330 000	
200	415 000	
(Coopers E 50 Loading)		

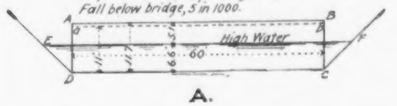
VII.

C.

Single Track				Double Track
Plate Girders Deck		Through Truss		Through Truss.
Span Ft	Weight Lbs	Span Ft	Weight Lbs	Weight Lbs
25	15,000	100	125,000	220,000
30	19,500	125	180,000	312,500
40	30,000	150	244,000	420,000
50	42,500	175	317,000	542,500
60	57,000	200	400,000	680,000
70	73,500	250	594,000	1,000,000
80	92,000	300	825,000	1,380,000
90	112,500	350	1,094,000	1,820,000
		400	1,400,000	2,320,000
		450	1,744,000	2,880,000
		500	2,125,000	3,500,000
		550	2,545,000	4,180,000

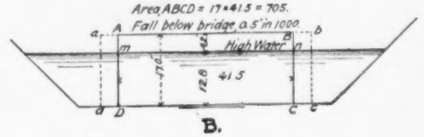
Fig. 3.

Drainage Area, 2.5 Sq. Miles; Slope, 35' in 1000'. Required Opening by table = 705 sq. Ft = ABCD. Fall below bridge, 5' in 1000.



A.

Area ABCD = 17 + 41.5 = 705.



B.

TABLES AND DIAGRAMS FOR RAILWAY LOCATING ENGINEERS.

C. P. Howard, M. Am. Soc. C. E.

ENGINEERING NEWS

A Journal of Civil, Mechanical, Mining and Electrical Engineering

Published every Thursday by

THE ENGINEERING NEWS PUBLISHING CO.

220 BROADWAY, NEW YORK

BRANCH OFFICES

CHICAGO: 1636 Monadnock Block
WASHINGTON, D. C.: Home Life Building
ATLANTA: Austell Building
RUDOLF MOSSE, BERLIN AND HAMBURG, GERMANY

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Entered at the New York Post-Office as Second-Class Matter.

The filtration of surface water supplies in Europe is general, and therefore it is no wonder that an English engineer should be shocked on learning that the people of Toronto discharge their sewage into Lake Ontario and draw their unfiltered water supply from the same source. The way in which such a proceeding and also the high consumption and waste of water strike an Englishman was set forth briefly in some extracts from the recent report of Mr. G. R. Strachan, M. Inst. C. E., on sewage disposal at Toronto, printed in our issue of Sept. 6. The remarks on water filtration and water waste prevention deserve to be given the prominence of repetition detached from the main subject of the report. They are as follows:

I cannot, however, close this report with comfort, without a digression to the water question. I know that voluntary advice generally meets with short shrift, yet I must express anxiety at the absence of filtration. With us filtration is so much used, even for our upland schemes, that its omission in the circumstances at Toronto has the effect of a shock on one's mind. The huge user of water undoubtedly operates as a hindrance to the adoption of filtration, by reason of the enhanced cost of the very large area of sand for filtering 100 gals. [120 U. S. gals.] per head per day, but notwithstanding that exaggerated burden I commend filtration as a prudent safeguard. Should the undue cost of the filters lead to the prevention of the waste now going on, so as to bring the financial burden within the usual limits, that would be a double advantage gained by filtration.

These words need no comment, but it may be remarked that many cities in the United States, as well as some in Canada, are also drinking their own sewage, in more or less dilute form, and may well give heed to Mr. Strachan's opinion of such a practice, particularly as he merely voices the consensus of opinion of the European countries that lead in measures for the conservation of the public health.

The association of dirt and disease as cause and effect has doubtless saved millions of lives since the dawn of civilization, and it is a pity that this relation is not more generally considered and acted upon. It is even more of a pity that the true relation between dirt and disease is not understood and made a guide to action, even by intelligent people. Evidence of gross misunderstanding of the elements of public sanitation is afforded by some recent press despatches, one of which, at least, has had wide circulation throughout the country. The most notable of these despatches was one which stated in some detail that the cause of an epidemic of typhoid fever at Three Oaks, Mich., had been found to be the nesting of large numbers of sparrows inside the village water-works stand-pipe. As our readers know, there is not the slightest evidence to show that typhoid fever could be spread by sparrows, and therefore such an assumption is likely to leave the real cause of a typhoid outbreak undiscovered and unremoved. The other and later instance is a report stating that a representative of the State Board of Health of Maryland had found on investigation that an outbreak of typhoid at Sharptown had been caused by "allowing pig-pens and outbuildings to go uncleaned." There is, of course, a considerable likelihood that the typhoid at Sharptown might be largely due to uncleaned and unguarded privies, inasmuch as it is now well established that flies play a very important part in the transmission of the disease. The significant feature of this despatch, and more particularly of the one relating to Three Oaks, is that the daily press of the country should pay good money for such despatches, with utter disregard as to whether or not they have any justification in fact or probability. The attitude of the daily press represents to a large extent that of the public mind, and both indicate the readiness with which people take alarm at imaginary dangers, when a calm statement of real dangers, of vital significance, would attract no attention whatever.

Air-line electric railways between New York and Chicago, with $10 fares and speeds of 125 miles an hour, are the latest features of stock-selling "enterprises," and are about as preposterous as the Tripler liquid-air scheme which was exposed in our columns some six years ago. There appears to be a bona-fide project for an electric railway between New York and Chicago, in which Mr. Joseph Ramsey (formerly of the Wabash Ry.) is interested, and which is said to be backed by foreign capitalists. But two other schemes are now being widely advertised, traversing the same ground and very nearly identical, except that one includes a branch to Boston. In response to inquiry, one company informs us that work is about to be commenced on its first section (an interurban line in Indiana); the other states that the project will be "pushed as rapidly as possible," but one of the gentlemen listed as a director writes us that he has no connection with the scheme. Whatever may be the merits of these two rival schemes, and these are extremely questionable (especially as only one line could be built), the methods of the projectors do not inspire any confidence, and the reprehensible feature is the way in which they are being exploited in order to sell stock, the methods being the same as those employed by liquid-air, Mexican plantation and other investment schemes. Full-page advertisements are inserted in leading daily papers, and although the statements are preposterous and misleading on the very face, these papers are willing to lend themselves to the financial exploitation of their readers as "gulls" and "suckers." We need not waste space in any consideration of the absurd claims and statements as to these railways, but the most objectionable feature is that the advertisements are specially designed to attract the public and the small investors who are likely to be influenced by the glaring statements, the specious promises, the ingenious references to investments by which fortunes have been made, and the fictitious estimates of traffic, revenue and cost of operation. In both cases it is the public and not the capitalist who is to invest, and the stock is advertised at $25 for a $100 share. One company appeals specifically to the "clerk, grocer, plumber, carpenter or other man of moderate means who has the foresight to realize the marvelous possibilities of this investment." The other company states that it will require over 5,000 experienced railway operatives, clerical employees, etc., and says "these positions will be open to experienced and meritorious men who have become stockholders in this company." Lottery companies are outside the law and their advertisements and literature are not allowed in the mails, and in the interest of the public it would seem that some steps should be taken to restrain these "investment" companies and put them under investigation.

The relatively insignificant quantity of water actually used for fire protection purposes is forcibly illustrated by some figures in the annual report of Mr. I. M. de Varona, M. Am. Soc. C. E., Chief Engineer of the New York Department of Water Supply, Gas and Electricity, in his report for 1905. Figures supplied him by the Fire Department show that the quantities used for fire purposes in the Boroughs of Manhattan and Brooklyn for the years 1900-5, inclusive, ranged from 49,033,000 gals. in 1902 to 99,229,000 gals. in 1901 for the Borough of Manhattan, with an average of 74,011,000 gals. for the five years, or the surprisingly small daily average of only 117,000 gals. In Brooklyn, the range was from 64,039,000 gals. in 1901 to 22,691,000 gals. in 1903, with an average for the five years of 43,706,000 gals. and a daily average for the period of 67,000 gals. A large percentage of the amounts given was drawn from the adjacent water fronts, going as high as 70% for Manhattan in 1901. This, however, is of course merely a local condition, but it renders all the more insignificant the quantity of Croton water used for fire protection. As Mr. de Varona states, even though the whole fire consumption in Manhattan for 1901 had been taken from the Croton supply, this quantity for the whole year (100,000,000 gals., in round numbers) would have been only about one-third of the total consumption for all purposes, for a single day. As is well said in the report, the difficulty in affording adequate fire protection is, not to provide the necessary volume of water, but to concentrate this volume at the scene of a fire. With this fact in mind, the new special fire protection system now being built for the more congested portions of the lower part of New York City is being designed to draw its ordinary water supply from the Croton system (although there will be emergency connections at the water front). But the two stations located at each end of the system will have numerous independent connections with large mains in different streets and will be provided with pumps having a capacity at each station of 15,000 gals. per min., or a daily capacity for the two stations of 43,000,000 gals. Provisions are made in the design for material additions to these quantities. With the proposed ultimate extension to a combined daily capacity of 69,000,000 gals., the volume potentially available would be in excess of the total quantity of fresh water used for fire purposes during 1903, in which year, of the five-year period cited, there was the heaviest draft upon the Croton supply.

While many engineers are well aware that the total yearly quantity of water used for fire protection is small, as compared with the total water consumption for all purposes, it is probable that very few who have not had direct experience in the premises are aware how little is used in connection with the largest fires ordinarily experienced in even a great city. Figures illustrating this point are so interesting and surprising that we give space for quoting them in full from Mr. de Varona's report as follows:

BOROUGHS OF MANHATTAN AND THE BRONX.

Oct. 29, 1900. Tarrant fire, 276-280 Greenwich St., of 22 hrs. duration; amount of water used, 330,000 gals.

Jan. 31, 1901. Wicks fire, 538-544 First Ave., 10 hrs. duration; amount of water used, 150,000 gals.

Feb. 22, 1902. 71st Regiment Armory, east side of Fourth Ave., between 33d and 34th Sts., 61 hrs. duration; amount of water used, 90,000 gals.

Dec. 21, 1903. 188-194 Mott St., 106 hrs. duration; amount of water used, 1,590,000 gals.

Mar. 26, 1904. 61 Broadway to 39-41 Trinity Place, 5 hrs. duration; 75,000 gals. of water used.

BOROUGH OF BROOKLYN.

Apr. 9, 1900. Eight 1-story, twelve 2-story, two 3-story, lumber-yard fire, S. S. Newtown Creek, 7 hrs. duration; amount of water used, 105,000 gals.

Apr. 30, 1901. 558-578 Flushing Ave., B. R. T. car sheds, 3 hrs. duration; amount of water used, 45,000 gals.

May 2, 1902. 239-245 Willoughby St., 5-story brick, 120 × 150 ft., Freeborn G. Smith Piano Manufactory, storage, 3 hrs. duration; amount of water used, 45,000 gals.

Nov. 30, 1903. 176-194 Montague St., 100 × 275 ft.,

Academy of Music, 2 hrs. duration; amount of water used, 30,000 gals.

Feb. 19, 1904. Two 2-story brick and frame, 2-story brick, 4-story brick, eight 5-story brick, Messrs. F. W. Devoe and C. T. Raynolds & Co., 3½ hrs. duration; amount of water used, 52,500 gals.

It will be seen that during even the Mott St. fire, which lasted for 106 hours, or 4½ days, only 1,590,000 gals. of water were thrown.

Aside from their specific bearing and interest, as indicated, attention may well be directed to the general relation of the foregoing facts to the distribution of the cost of a public water supply between the public and the private consumers. While it is true that the actual consumption of water for fire purposes is insignificant, yet large and long conduits as parts of gravity supplies, large pumping stations where gravity supplies are not available, large reservoirs in many instances and large distributing mains and laterals in all cases, in order to be able to concentrate upon a fire a volume of water far in excess of the domestic consumption, all combine to swell the capital charges of a waterworks system, even though the operating expenses on account of fire protection are of little consequence. In view of this indisputable fact, justice demands that a considerable sum of money should be raised from the owners of property benefited by fire protection, to pay the cost thereof, instead of assessing the whole charge upon water consumers.

THE INVESTIGATION OF OLD BRIDGES: A PHASE OF MAINTENANCE ENGINEERING.

The valuable paper of Mr. W. J. Watson, M. Am. Soc. C. E., on Investigation of Old Bridges,* which opened the season of the American Society of Civil Engineers last week, deserves fuller appreciation than seemed to be accorded it by the almost nominal discussion at the meeting. Its most prominent merit lies in the fact that it deals with a phase of an almost wholly neglected subject. *Maintenance engineering*, if we may be permitted to coin a term, has no more responsible and delicate a duty than the determination of the point of time when a given structure becomes unserviceable and requires renewal. The performance of this duty includes the subsidiary problem of fixing the service value of a given structure; for instance, in the matter of strength, deciding whether a structure is safe for specified service and approximately what margin of reliance it has. That it is a responsible and delicate duty will no doubt be readily apparent on reflection, but a reference to the money considerations involved will make the fact more emphatic. In the design of a new structure it is not a very serious matter, in point of cost, whether loads or permissible stresses are assumed 5% higher or 5% lower, or even 10%; the proportion of added cost of the "richer" design will be *at most* the same percentage as the decrease of unit stress, and in general it will be far lower. In the investigation of an existing structure, however, a 10% difference in allowed stress may mean the approval or the condemnation of the structure; in other words, it involves a difference in expenditure equal to the entire cost of renewal. In the first case, a new structure, the "cost" of a 10% variation in unit stress may be 5 to 10% of the total value of the structure; in the case of the old structure, an equal variation in the allowed stress may involve an expenditure exceeding 100% of the cost of a new structure, which equals possibly 150 or 200% of the inventory value of the old structure itself. Thus, in the latter case it is just about twenty times as important to fix the unit stresses properly. It follows that a very much sharper line must be drawn between safety and danger; loads must be accurately fixed, stresses and secondary effects fully determined, and the highest unit-stresses which can be safely allowed must be adopted for the calculation. Mr. Watson, in opening to discussion an important field of the important subject of maintenance investigations, has made a distinctly meritorious contribution not merely to bridge engineering but to engineering in general.

A quite different phase of the same subject was discussed several years ago* in a paper read before the American Society of Municipal Improvements, under the title "The Life of Pavements," by Mr. Geo. W. Tillson, M. Am. Soc. C. E. In that paper the author pointed out that at a certain period in the life of some kinds of pavement the expense of maintenance becomes excessive, and it is then cheaper to tear up the old and lay a new pavement; an attempt was made to determine this period by a formula.

Bridges and pavements differ markedly in their relation to maintenance and renewal. Pavements call for large current repairs, bridges for little or none; the deterioration in the latter is small, while in pavements it is very large. On the other hand, pavements can be made virtually as good as new by repairing, whereas a bridge can not. Its small but progressive deterioration on the one hand, and the change of loads and other conditions which it is subjected to, ultimately make replacement necessary for definite reasons of safety. The complete renewal of a pavement (macadam or asphalt, the specific pavements to which Mr. Tillson's argument applied) is largely an economic question, and is necessitated by the fact that while the repairing of a pavement makes it virtually like new as concerns immediate excellence, its durability cannot be brought up to the original measure; that is to say, the maintenance cost increases with every year. Thus the pavement presents, from the maintenance standpoint, mainly a problem of financial expediency; the bridge, on the other hand, presents a problem of adequacy for definite service, almost independent of economic considerations.

In these two instances are represented two distinct aspects of maintenance, in almost pure application in either case. The great majority of maintenance problems involve both aspects, more or less intricately combined. Railway maintenance and line improvement offers a good illustration of this. To determine the proper standard of excellence of track for a given piece of railway line, to decide when double-track, cut-off-lines, grade-reductions, etc., are demanded, when yard and terminal improvements should be undertaken, when better motive-power should be procured, how thorough and frequent the inspection of engines and cars must be, are all questions wherein adequacy and cost are closely interwoven. These questions are nearly always solved by that obscure, unconscious process of reasoning which results in a decision based on so-called "judgment." Few of them have ever been subjected to general analysis, with a view to making their solution in any special case easier or more accurate. Yet it seems certain that all or nearly all are susceptible of analytical study as general problems.

Returning to the matter of strength or adequacy, the phase with which Mr. Watson's paper is solely concerned, we find a highly important proposition in the subject of old structures clearly stated in that paper. It is this: that standard specifications for new structures are far from being proper guides in the analysis of old structures. The reasons which establish this proposition are somewhat difficult to set forth; at least a considerable space would be required to express them fully and clearly, but they may be followed out by the individual engineer if he reflect on the purposes of a working specification, and the factors which determine its clauses. A specification is not an abstract statement of principles and constants of design; it may contain some elements of such a statement, but its essence is of different character. The leading purpose of a specification is to serve as a rigid circumscribing barrier to the mixed motives and mischances of design, manufacture and erection, compelling the action of these incidentals to remain within such limits as will produce a safe structure as desired. The specification is a legal document, a contract clause, and differs from the ideal purpose involved in much the same way as the text of a marriage certificate differs from the true spirit of the agreement. On the other hand, in the investigation of an old structure no clauses need be followed whose main purpose is to guard against the "skinning" proclivities of the designer or the detailer; no club clauses are applicable; no contingencies of injury or imperfection in the shop fabrication and erection need be contemplated; and, passing from negative to positive characteristics, the investigator may take into account elements of deficiency or excellence which specifications do not touch, as (in a steel bridge) rigidity, good stress distribution at panel-points, symmetry, continuity, and the like. Engineering acumen and trained judgment find preeminent application here.

In a vital detail of the recommendations which Mr. Watson makes for allowable stresses in old bridges, we believe he goes beyond a safe mark through not taking into account certain special contingencies which affect this detail. We refer to his advocacy of much higher stresses for rivet connections than are customary. His contention is that rivet stresses in current practice are taken too low, principally because of non-inclusion of plate-friction. Conceding this contention for the sake of argument, we cannot grant its unconditioned applicability to the investigation of old bridges. It is a notorious fact that the bridge and structural work of years gone by abounds in poor detailing, and that through inadequate or neglected analysis of details the latter were not up to the standard of design of the main members. The principle that details should always be proportioned for the full strength of the member which they serve, now a generally accepted principle, found little recognition in those days. The resulting defects in details, as far as they can be detected by external examination, can be allowed for, without prejudice to high unit stresses, and Mr. Watson doubtless intends this. But it is by no means certain that all defective products of poor detailing appear externally, and this contingency must be thrown into greater margin on the unit stresses. Further, the workmanship of riveting was never of higher grade than it is to-day. Power riveting, more uniform material and more closely-regulated practice in shop and field, give the riveting of to-day a marked superiority to that of former days. And still more important in its application to old bridges is the fact that a bridge under investigation may be of unknown origin, or may have been built under conditions tending to lower the efficiency of the riveting, resulting in a considerable number of burned or poorly-driven rivets. Altogether, the elements of uncertainty in the rivet work of an old structure are greater than those which affect the main sections. Correspondingly a greater margin must be allowed; the unit-stresses derived from general reasoning must be reduced by a larger factor in the rivet stresses than in the main stresses. It seems necessary to conclude that the relative stresses for details and main members accepted in modern practice should not be materially departed from in investigating old bridges, or that the details should be allowed lower rather than higher stresses (in relation to the unit-stresses of the main members) than would be applied to a structure built under modern conditions.

*Engineering News, Aug. 23, 1906, p. 200.

*Proceedings of the American Society of Municipal Improvements, 1899, p. 133.

LETTERS TO THE EDITOR.

Regarding the Hollingsworth Underflow Ditch.

Sir: Having read Prof. Slichter's description, in your issue of July 5, 1906, of my underflow ditch, situated near Ogalalla, Nebr., I deem it my duty to differ in some respects with the author. In regard to the description of the ditch and the amount of water developed, I have no objections to make, but I think some of the conclusions are wrong.

I have operated the ditch for the past ten years, with very little expense, except for one year, when the overflow got into it and of course stopped the seepage. That, however, was my own fault, as my dike was not high enough. I opened the ditch up and raised the dike, and

I think there will be no danger from that source in the future.

The statement that the ditch fills with sand is erroneous. Except for the above-mentioned instance, sand has not into the ditch only from the tramping of cattle on the banks. That could all be prevented by fencing the ditch.

I find the expense of operating a seepage ditch, in the case of a sandy stream, is much less than that of an open ditch, since in the latter sand is continually being washed in by the flow of water and requires re-moval every year. With a seepage ditch, also, there is no expense for headgate and the flow of water is uniform no matter which way the wind blows. Consequently there are no overflowing laterals, with water flowing where it is not wanted.

Another advantage of a seepage ditch is in the matter of winter irrigation. I have carried water through my ditch during the coldest winter weather of the past ten years. For ordinary crops this irrigation, with what rainfall we have here, has been sufficient. Another advantage connected with winter irrigation is the lack of expense for spreading water; all you have to do is to turn the water into the laterals and it will do its own spreading. Besides, the freezing of the water lifts the soil and loosens the ground for spring work.

In conclusion, let me say that, considering the cost of the machinery of a pumping plant, the fuel expense, and the wages of a man to operate it, a seepage ditch is so far ahead that there ought to be no comparison. A Mr. Auftengarten, living northeast of Ogalalla, on the North Platte River, bought a pump six or eight years ago, paying $2,000 for it. It consumed two carloads of coal, which he had shipped from Rock Springs, Wyo., and succeeded in wetting his ditch for a distance of a mile; but it never furnished enough water for any irrigating. The pump stands to-day on the bank of the river unused.

In my opinion, the entire South Platte valley can be irrigated from seepage water. The amount of water obtained depends upon the length and depth of the canal and the kind of sand it is constructed through; the coarser the sand, the more water.

A. Hollingsworth.

Ogalalla, Nebr., Aug. 3, 1906.

Artificial Basin and Tide Locks for Ancon Harbor, Panama Canal.

Sir: The plan adopted by the present Isthmian Canal Commission and endorsed by Act of Congress, as respects the Pacific Division of the Panama Canal, contemplates a waterway of enlarged prism and lock dimensions, with an artificial lake having an extent of eight square miles, that would be formed by an extensive system of massive earthen dams. Double locks in flight are to be built at Sosa, near the northern apex of Panama Bay, or Ancon Harbor. The northerly end of the lake extends to Pedro Miguel where another pair of locks leads to the summit level at elevation 85 ft. From Pedro Miguel to Sosa, a distance of 5.4 miles, the navigable channel through Lake Sosa has a breadth of from 500 to 1,000 ft. The distance covered by the Sosa locks is 0.4-mile; and the channel thence lays in a southeasterly direction, with some curvature, through the upper part of Panama Bay, a distance of 4.25 miles, where it terminates, the normal depth of the sea at that point being 45 ft. The full length of the Pacific division is therefore approximately ten miles.

The channel in Ancon Harbor for two miles of its extent will occupy the site of the existing ship channel that has been dredged to a least depth of 16 ft. for average low water stage, which is equivalent to 22 ft. as referred to mean sea datum. The same is to be deepened to 45 ft. and given a uniform bottom breadth of 300 ft.

The complete impounding of the waters of Ancon Harbor by means of dikes, so constructed as not only to hold back the sea from sweeping in, but to prevent the waters in the bay from flowing out, would make it practicable to extend the artificial level as far as Perico Island, where would be located the tide locks. The level of Ancon Harbor could then be maintained constantly at elevation 9 ft. above mean sea level, and the channel would be excavated to a depth of 40 ft. below that plane. It should be remarked that the range of the tides is 3 ft. less at Perico Island than at La Boca or the head of Panama Bay. Therefore high tide must be taken as 8½ or 9 ft. instead of 10 ft. above mean sea datum.

The configuration of the shore, the triangular shape of the bay with the two points of land jutting out prominently on each side, the positions of the little group of islands, about equally distant from these two salient features of the coast line, and the shallow waters are all favorable conditions for the building of two long moles connecting the islands with the mainland. The two islands, Naos and Perico, being then joined by another embankment, the separation of the waters of Ancon Harbor from the sea outside would be complete. The dike on the northeast side of Perico Island, extending from Panama Point, would of course end at the locks, which would be built of the prescribed dimensions, together with the approach walls leading thereto at each end. For such works, the locks and leading piers, the dikes would be extended on the side adjacent to the island, and also on the northeast side of the area which would contain the locks. Including the approach walls this space would be necessarily about 2,000 ft. long and 300 ft. wide, being shut in at the southerly end by an ocean bulkhead, no work of this kind being required at the northerly end of the lock area. By permitting the waters of the bay to flow out of the impounded basin at low tide through iron pipes or regulating gates of suitable and convenient arrangement the level of the water inside could be brought down nearly to the lowest level of the tide. Centrifugal pumps would then be installed having an aggregate capacity of 200 cu. ft. per sec., or 15,000,000 cu. ft. of water per day.

The next step would be to turn the Rio Grande River from its course by a diversion channel. The inflow from the Rio Grande to the enclosed harbor being thus disposed of, and the water surface of Ancon Harbor being but little more than two square miles in extent, at low tide, and having an average depth of no more than 10 ft., would contain approximately 600,000,000 cu. ft. of water; all of which volume could be pumped out in less than 60 days. The pumping plant would be further utilized to keep the harbor free of water at all times during the period of constructive operations.

After the impounded basin has been unwatered the deepening and excavation of the channel would proceed in the same manner as now put in practice by contractors on the West Neebish channel in the St. Mary's River. Channeling machines are employed and excavators mounted on trucks dig out the broken rock, which is in turn emptied into skips and carried away to the spoil bank on cableways. The work instead of being submarine would be by the dry process, with a corresponding reduction in cost. That is to say, for rock excavation the difference would be as between $2.50 and $1 per yd. With so much lessening of cost for rock removal the channel could be made 400 or 600 ft. wide, as might be considered desirable. For the channel as proposed, 300 ft. wide and 40 ft. deep below the plane of elevation (+ 9 ft.), there would have to be excavated approximately 600,000 cu. yds. of rock and 1,500,000 cu. yds. of earth, the collective cost of which would be $900,000.

An efficient and economical mode of breakwater construction is an essential consideration in the solution of this problem. In the building of the moles there would be a length of 1½ miles from Guinea Point on the west side of Panama Bay to Naos Island, also a length of 1¾ miles from Panama Point to Perico Island; also a dike ¼-mile long between the two islands, Perico and Naos. To this must be added 0.6-mile of dike construction and ocean bulkhead to enclose the cofferdam space of the Perico locks. These dikes could be made 50 ft. wide at the top, the same being raised to a level of 19 ft. above mean sea datum, and the sides would be finished to a slope of 1 to 2. As compared with a levee construction, the breadth at mean tide level would be 125 ft. from side to side. Storms never come from the south or east at Panama, and a sea wall of massive design to oppose the force of boisterous winds and seas is non-essential. A riprap of coarse rock and rubble thrown upon the outer side would serve to break the force of the waves. The breadth of the dike, 90 ft. at high tide level, and the mass being composed of mixed coarse and fine material, are depended upon to prevent seepage or percolation in any considerable or dangerous amount.

This plan of construction would imply and necessitate a rearrangement of the Sosa and Pedro Miguel locks, and a modification of the level of Sosa Lake. Instead of two locks in flight at Sosa there would be a single pair of locks having a lift of 38 ft. The Pedro Miguel locks would also have a lift of 38 ft. instead of 30 ft. The level of Sosa Lake would be at 47 ft. above mean sea level instead of 55 ft.

From south to north the levels and locks would stand as follows:

Perico lock, — 9 to + 9 ft.	18 ft.
Sosa lock, 9 to 47 ft.	38 ft.
Pedro Miguel, 47 to 85 ft.	38 ft.
Total	94 ft.

The La Boca and Sosa-Ancon dams could be reduced in height from 80 to 64 ft.; because of diminished head pressure, amounting to 27 ft., and greater stability and safety would also be secured. The crest of the Ancon-Corozal dam could be lowered from 80 to 72 ft., a difference of 10%. By diminishing the size of these heavy embankments the cost would be reduced by about 40% for the two southerly dams and by 20% for the easterly dam. The number of locks would be the same, but there would be three separate sets of locks, instead of there being two pairs of locks in flight at Sosa.

The cost of the Pacific Division of the canal with an artificial level in Ancon Harbor may be computed as follows:

Pedro Miguel locks	$7,500,000
Approach walls	350,000
Pedro Miguel to Sosa locks	1,000,000
Sosa locks	7,500,000
Approach walls	400,000
Diversion channel and regulating works at Ancon	200,000
La Boca dam	1,000,000
Sosa-Ancon and Ancon-Corozal dams	1,150,000
Channel in Ancon Harbor and Panama Bay	900,000
Perico locks	5,500,000
Approach walls	400,000
Dikes, auxiliary works and harbor improvements	2,650,000
Total cost	$28,550,000

The net additional outlay for the proposed Ancon Harbor improvement, as compared with the estimates of the Canal Commission covering the Pacific Division, would be approximately $1,766,000. The compensating advantages may be enumerated as follows:

(1) A commodious harbor perfectly protected on all sides. The complete submergence of the mud flats about two square miles in extent which are bare at low tide, a prolific source of disease.

(2) The tide lock is not vital for any scheme of defense in time of war. If disabled, the waterway would still be navigable, its entrance having the same depth at high tide. The locks at the proposed location near La Boca would be exposed to the fire of a hostile fleet or to attack by torpedo boats or other submarine engines of warfare. The two long moles would afford a large measure of defense, inasmuch as the fleet or boats of the enemy could not approach nearer than a distance of two miles.

(3) From Sosa locks to beyond Perico Island, a distance of 3.5 miles, the navigation would not be disturbed by tidal currents or sea-way as at present. Easterly currents sweeping across Panama Bay tend to fill up the channel with sand and silt and dredging must be resorted to constantly. The southerly dikes would obstruct such currents and no further undue expense for dredging would be necessary.

(4) Beacon lights being placed upon the ends of the piers, which constitute the approach walls to the Perico lock, would afford a plain guide for mariners coming into the canal entrance with their ships at night. While the impounded area of the bay is unwatered proper towers could be erected to mark the ship channel both during the day time and by lights at night.

(5) If it shall be ascertained, after more complete data have been gathered and more careful estimates have been made, that the excavation in Panama Bay will cost $4,500,000, or to exceed $3,750,000, which is but little more than half the assumed cost of the same channel in the sea-level project, the factor of economy would weigh in favor of an artificial level in Ancon Harbor.

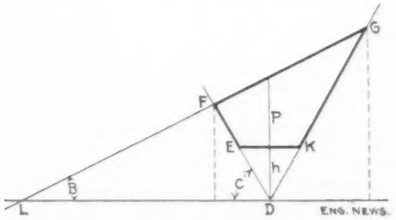
Yours truly, Lewis H. Cowles.

581 Arcade, Cleveland, Ohio, Aug. 31, 1906.

On Computing Areas of Cross-Sections.

Sir: The letter from Mr. Elbert O. Brower, in your issue of Aug. 16, and that of Mr. F. Lavis, in the issue of Aug. 30, reopen a much discussed though always interesting subject. It is interesting because of the many different solutions of the same problems, which are usually simple and yet involve some very pretty geometrical and trigonometrical relations; and such discussion may be profitable even if it does not always lead to valuable practical results.

Mr. Brower has derived a formula for that case where the center height and ground slope are known. This is the data that may be taken from profile and contour map or topographical notes, and is such as is used in making preliminary estimates. The well-known formula quoted by Mr. Lavis is that for use where the cross-section notes are known, and is therefore entirely foreign to the case brought up by Mr. Brower.



Mr. Brower may care to compare with his formula the following: On the accompanying figure his symbols are used and are plainly indicated. The area EFGK = A is desired, and area DEK = Q is known for any given roadbed and side slopes.

$$\text{Area } A + Q = LGD - LFD = \frac{(h+p)^2 \cot C}{1 - \cot^2 C \tan^2 B} \qquad (1)$$

In cases where side slopes are known (as in practice) this formula may be simplified. Thus, taking C = 32° (as in Mr. Brower's example) the formula becomes

$$A + Q = \frac{(h+p)^2}{.625 - 1.6\tan^2 B} \qquad (2)$$

while Mr. Brower's formula for the same case becomes

$$A + Q = \frac{(h+p)^2(\cos 2B + 1).4494}{\cos 2B - .4384} \qquad (3)$$

Formula (2) is seen to be much simpler than (3), although the latter has an advantage in the use of the double angles.

For level ground either formula reduces to

$$A + Q = (h+p)^2\ 1.6 \qquad (4)$$

In determining ground slopes from topography notes or from a contour map, the angle of slope is usually expressed by the ratio of vertical to horizontal distances. If for a rise of one unit we have x horizontal units formula (1) becomes

$$A + Q = \frac{(h+p)^2 \cot C}{1 - \frac{\cot^2 C}{x^2}} \qquad (5)$$

and formula (2) becomes

$$A + Q = \frac{(h+p)^2}{.625 - \frac{1.6}{x^2}} \qquad (6)$$

In actual practice one would use such formulas only as a basis for preparing diagrams or tables, to be used in connection with tables of level areas which should be available. To determine the area to be added to the value of the level area taken for the given center height one need only subtract (4) from (6), giving

$$a = \frac{1.6(h+p)^2}{.39x^2 - 1} \qquad (7)$$

which gives the amount by which we must increase the level area to obtain the area of section with slope indicated by x units horizontal to one vertical. Comparing this with (4) we note that the total area is obtained by multiplying the level area by $1 + \frac{1}{.39x^2 - 1}$ from which we may prepare the following statement:

For slope 1:3 total area = level area multiplied by	1.40
1:4	1.20
2:11	1.10
1:6	1.08
1:7	1.06
2:15	1.05
4:34	1.04
2:19	1.03
etc.	etc.

In his own work of this nature the writer has preferred to use tables of level areas supplemented by a diagram made entirely of straight lines, for determining the heights of "equivalent level sections" in which case the quantity "Q" need not enter into the calculations.

Respectfully, Erdis G. Robinson.

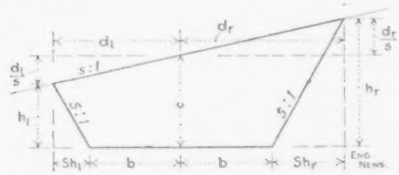
515 West First Ave., Columbus, Ohio, Aug. 31, 1906.

Sir: I am taking the liberty of indulging in a display of "mathematical fireworks" in order to present a simple factorial formula for computation of cross-sections in preliminary estimates that in my opinion covers a phase of the subject not covered by those that have recently appeared in your correspondence columns. The familiar formula cited by Mr. Lavis in the issue of Aug. 30 is of course in an entirely different category, being applicable to final location rather than to a preliminary estimate based on a paper location; and I hope that he will not be "misled" into attempting to use this formula for three or five-level ground.

The derivation of the formula from the familiar

Area = ½ [center cut × sum of distances out + ½ roadbed × sum of side heights]

is as follows:



In the figure c = centre cut; b = ½ roadbed; s : 1 = slope of ground surface; S : 1 = side slope d_l, d_r, distances out, left and right; and h_l, h_r the corresponding side heights.

As above,

$$A = \tfrac{1}{2}\,[c \Sigma d + b \Sigma h].$$

By inspection of the figure:

$$d_l = b + Sh_l \qquad h_l = c - d_l/s = \frac{sc - b}{s + S}$$

$$d_r = b + Sh_r \qquad h_r = c + d_r/s = \frac{sc + b}{s - S}$$

$$\Sigma d = 2b + S\Sigma h \qquad \Sigma h = \frac{2\,(s^2 c + Sb)}{s^2 - S^2}$$

whence, by substitution and collecting terms,

$$A = \frac{S}{1 - \frac{S^2}{s^2}} \left[c \left(c + \frac{2b}{S} \right) + \frac{b^2}{s^2} \right].$$

The coefficient involving the slopes is readily deduced by direct computation or by the expansion

$$\frac{1}{1 - a^2} = 1 + a^2 + a^4 + \ldots$$

I have found the following table very convenient; of course s may be expressed in per cent. if desirable, and any further values of s and S may be introduced with but little labor.

Table of values for

$$K = \frac{S}{1 - \left(\frac{S}{s}\right)^2}$$

Side slope S:1 [ft. horizontal to one vertical].

Ground slope.

s : 1	1 : 1	1½ : 1	2 : 1	½s²
∞ : 1	1.0000	1.5000	2.0000	.0000
100 : 1	1.0001	1.5003	2.0008	.0001
50 : 1	1.0004	1.5014	2.0032	.0004
40 : 1	1.0006	1.5021	2.0050	.0006
30 : 1	1.0011	1.5038	2.0088	.0011
25 : 1	1.0016	1.5054	2.0128	.0016
20 : 1	1.0025	1.5084	2.0202	.0025
15 : 1	1.0044	1.5152	2.0360	.0044
10 : 1	1.0101	1.5345	2.0834	.0100
8 : 1	1.0158	1.5546	2.1333	.0156
6 : 1	1.0286	1.6000	2.2500	.0278
5 : 1	1.0417	1.6484	2.3810	.0400
4 : 1	1.0667	1.7454	2.6667	.0625
3 : 1	1.1250	2.0000	3.5998	.1111
2 : 1	1.3333	3.4285	∞	.2500
1 : 1	∞	.....	.....	1.0000

The use of the formula is demonstrated by the following simple example:

Let b = 10, c = 12, s = 20 : 1, S = 1½ : 1. Then $\frac{2b}{S} = 13.33$ and $b^2 = 100$ will be constant for a considerable section of the work. Whence $A = 1.508 \times 12 \times 25.33 = 459$ sq. ft. performing the multiplications by slide rule and neglecting the final small term (= 0.38 sq. ft.). It is convenient to place a check mark in the column of $\frac{1}{s^2}$ opposite the value of s which makes the term $K \frac{b^2}{s^2}$ appreciable.

While I have never seen this formula elsewhere, I cannot be assured that it is original, but merely present it with the hope that it will not be out of place.

Yours truly, F. L. Humphrey.

Eng. Aid, U. S. Reclamation Service.

Snyder, Okla., Sept. 1, 1906.

Sir: In your issue of Aug. 30 Mr. F. Lavis criticises the formula of Mr. Elbert O. Brower (Engineering News, Aug. 16, 1906) on areas of railroad cross-sections. Mr. Brower in his letter referred to Mr. Sloan's method given in Engineering News, June 28, 1906.

No one who has read Mr. Sloan's letter intelligently could suppose that it was intended to calculate areas from cross-section notes by his methd. It would of course be absurd to use Mr. Brower's formula for that purpose. The letter of Mr. Sloan read in the same connection may serve to "elucidate" that of Mr. Brower. The application of such a formula is, as explained by Mr. Sloan, to make tables of quantities for different ground slopes and side slopes, for use in approximate estimates, where only the center cutting, or filling, is known, together with the slope of ground. This information is obtained from the profile and contour sheets.

The method of computing such tables, when ground is level, is shown in Trautwine, p. 790 (1902); that is, by first and second differences. Where tables are to be calculated for different slopes of ground, 20 on 100, 30 on 100, 40 on 100, etc., the same method applies. The second difference will be represented by twice the sum of the areas of the two triangles, a n o (see Trautwine), one on each side of the figure. These triangles will be oblique instead of right angled. Of course the method fails where the cut or fill is so small that the cross-section represents a triangle with base less than the width of the roadbed. The difference and second difference can be obtained at once by calculating three consecutive areas, as explained by Mr. Sloan, carrying the figures out to three or four decimal points. It is of course necessary to calculate them accurately as any error in the second difference increases about as the square of the number of times it is used in series. Nevertheless the tables can easily be written out by these differences to three or four decimal points, checking up say every tenth result by a straight calculation of the area. Each area and difference should in the beginning be reduced to cubic yards so as to make out table in cubic yards rather than in square feet.

The writer has in his possession a set of 14 tables of this kind, each one different as to slope of ground, width of roadbed, or side slopes. Some of them run up to a height of 75 ft. It would be quite an undertaking to calculate each consecutive area of every one of these tables by the regular cross-section rule referred to by Mr. Lavis, namely, $A = \frac{h_1 + h_2}{4} R + \frac{d_1 + d_2}{2} G$.

The labor would be increased by the fact that each one of the side heights and side distances is unknown, and would have to be platted or calculated. If calculated, it involves the solution of two oblique triangles for each cross-section, and if calculated accurately enough to make use of the method of differences, it will be found that the good old formula, so applied, is not so simple after all.

Of course, as an approximation, one area and the cubic yards for about every 5 ft. of variation in center cut (or fill) can be calculated from a cross-section sheet by the ordinary formula, and the intermediate quantities interpolated by simple proportion. The error will probably not exceed 4% on any ground slope under 40 on 100, and will be close enough for approximate estimates. This is one way to do it.

Mr. Brower in his method uses the roadbed triangle bounded by the roadbed, and the side slopes produced. The ordinary formula does not. When slopes are taken from contour sheets, the slope of ground can be conveniently represented in the form of a fraction $\frac{\text{rise}}{\text{base}}$, = G; and side slopes by a similar fraction = S. Thus a ground slope of 40 on 100 would be G = 0.40 and a side slope of 1½ to 1 or 2 on 3 would be S = 0.67. Calculating the side heights and distances by straight line equations (as the easiest method) and figuring the area either by the ordinary formula quoted by Mr. Lavis, or by using the roadbed triangle and slopes produced, we obtain the same formula:

$$\text{Area} = \frac{S\,(4\,h\,R\,S + 4\,h^2 + G^2 R^2)}{4\,(S^2 - G^2)}$$

where S = side slope; G = ground slope; R = width of roadbed, and h = center height (depth of cut or fill). For any given table where the ground slope, side slope, and roadbed are the same throughout, the equation can, if desired, be reduced to the form; Area = $B\,h^2 + D\,h$ + K, where B, D and K are constants, and h, the center height, is the only variable. It is not claimed that the above formulas has much advantage, if any, over those of Messrs. Sloan and Brower. It is not well adapted to the use of logarithms.

Yours truly. C. P. Howard.

Empire Bldg., Pittsburg, Pa., Sept. 5, 1906.

Some Comments on "Waste of Water in New York and Its Reduction by Meters and Inspection."

Sir: I have just read the article in your issue of Aug. 9, based on Mr. James H. Fuertes' report on "Waste of Water in New York," etc. With some knowledge of what such reports usually are, and may be, it is a pleasure to observe that this is a piece of work logically arranged, unbroken in argument, bountifully supplied with proof—to sum it all up, a piece of work well done. It is nigh ungracious to write at all in criticism of such a report, and nothing but the never-ending struggle for perfection can justify it. With more time at the disposal of the author, no doubt he would himself have hit upon some, at least, of the improvements to his report here to be suggested. They are given with a full appreciation of the author's own plainly-to-be-recognized endeavor to spare no pains to make his report as good and complete as hard work on his part could make it.

The main conclusion, that it would pay, and pay well, in New York City, as elsewhere, to convert waste—or leakage, if the author prefers to call it so—into water that is used, is unassailable. But the proof could, I submit, have been made to show a greater possible saving of water.

The argument of the report, as a whole, is as follows: A reasonable percentage of unaccounted-for water, assuming the city to have been completely metered for several years, is 25%. This is the educated judgment of the author; not the result of experience or computation. But if made only 20%, instead of 25%, it would at once make the percentage of the total supply that may probably be saved by stopping all house leakage 20% instead of 15% (p. 62), with an enormous effect on the conclusions of Part IX., "The Introduction of Meters in New York as a Business Proposition." The further division of the 25% of the supply, of leakage and wastage, into a 15% that is "effectively and quickly controllable," and into a 10% that is beyond redemption, is only an arbitrary one. The author proves that it will well pay to convert the 15% named from waste into use; and it should only be holding up his hands in the good work to show that in saving this 15% he may be building better than he knew, and to show further that if properly and permanently done the saving may result in finally reducing waste to some 5 gals. per capita.

In my opinion, all expressions of leakage in percentages of the daily supply are wrong in principle and misleading. Leakage is a function of the pressure in the mains, and of the number of miles of main and service pipe and of the skill and amount of care and conscience at the time and for many years previous exercised to prevent, find and remedy leaks and waste. Leakage has nothing whatever to do with the habits of the water-takers and the amount of water they use or consume per capita. Its proper expression is in gallons per mile of main and service pipes; or, if it be assumed that the

proportion between miles of main and service pipes and number of inhabitants or water-takers is nigh constant, as it is, leakage may properly also be expressed in gallons per capita.

This unfortunate flaw in the author's argument greatly vitiates the conclusions drawn in the report from his otherwise admirable Table 12 [see Engineering News, August 9, 1906], as the table itself fully indicates. Thus, a leakage of 8.7 gals. per capital at Fall River puts that city in the same class with Harrisburg, which has a leakage of 7 gals. per capita, or more than three times as much; and also in the same class with Quedlinburg, having a leakage of only 1.5 gals. per capita, about one-sixth of the leakage of Fall River; and it also classes together Harrisburg and Quedlinburg, the first of which has a leakage twenty times as large as has the last. Comparison similarly Hackensack, leaking at every pore to the extent of twenty times the leakage of Fall River, and yet rated in the table at only twice the allowable leakage, or in the same class with Wellesley, which leaks only one-eighth as much as Hackensack. The difference between reckoning per water-taker and reckoning per capita will modify, but will not destroy this last comparison.

Now, leakage is almost wholly a matter of lack of proper watchfulness, in the past and in the present; the difference between hard work and shiftlessness, more or less long-continued, or more or less frequently recurring, and cannot be represented by a rated uniform percentage of consumption. On p. 119 the author himself says that "the only really practical indication of excessive street main leakage is to be found in the *amount* [Italics mine] of unaccounted-for water."

The sources of such leakage are given on p. 107: (1) Leakage out of mains; (2) leakage of service pipes (see also pp. 148 and 164); (3) valves and corporation cocks; (4) hydrants; (5) unknown connections, that is, stolen (see also pp. 113 and 115); (6) errors of measurement. *Broken* mains discharging into sewers or other underground channels might have been added.

The removal of all these causes is possible with good husbandry, and, as Table 12 shows, leakage has thus been reduced as low as 8 gals. per inhabitant in this country and to 1 gal. per inhabitant in German cities.

So soon as the amount of such leakage is recognized as dependent merely on the quality of the work which is put into operation, foreign cities and small cities can be directly compared with New York or with other large American cities. New York should have a much smaller leakage out of mains and services than most cities, even with no greater amount of defects in its distribution plant, because the pressure in New York has always been comparatively low; but the reverse is the case.

Heeding the author's suggestion (p. 20) that the experience of other cities in which the problem has been attacked is the best guide to what may be accomplished, let us cite two of the few available home examples. Into the pipe lines of the East Jersey Water Co., built by and eleven years in charge of the writer, some 40 miles in extent, running under and over rivers, through swamps, up hill and down dale, in some places under 400 ft. of head, the water was received through two meters and distributed from them through eleven other meters. The discrepancy of the daily accounts of these two groups of meters was never allowed to be more than 0.5%, say 2 or possibly 3 gals. per capita. So soon as the leakage jumped beyond this mark, and it usually went with a jump, we started out to find, found, and repaired the criminal leak. In the Metropolitan Water District of Massachusetts the same quality, possibly a still better quality, of management is now being shown. Time and again the chart recorders of their meters have revealed to the employees a main pipe broken over night, or other undue leak occurring, and have been the means of the immediate stoppage of that leak, which otherwise might be running yet and continue to run indefinitely, unknown and unsuspected.

If New York undertakes the redemption of the water now wasted out of her supply it should be done with the intention of never again abandoning the keeping of *daily* chart records of all water received and again distributed to the established sub-districts. Income and outgo must be known and watched continuously. In this way leakage may be reduced to and maintained at probably in the vicinity of 5 gals. per capita, where it now is 62 (p. 62), and where the author has fixed upon 25 gals. as impossible of recovery for all time. Unquestionably, many leaks not classed as preventable will be discovered and stopped while the work goes on of stopping those classed as preventable, and it would be a waste of work accomplished to allow such to regain their former power of making inroads on the supply at hand by omitting to keep up a continuous daily watch on all water income and outgo. This precise thing has been done for fifteen years on the works built by the East Jersey Water Co., and these three or four years by the Metropolitan Water Board of Massachusetts, with gratifying results.

The Merchants' Association committee says, indeed (p. 4), that "defective mains and service pipes abound in all cities, and it is a violent assumption that they are more general in New York than elsewhere"; but water-works superintendents and engineers who have had experience here as well as elsewhere will hardly agree to this patriotic proclamation. In the first place, the New York water-works are older than the average, and that alone makes defective mains and service pipes much more general in New York than in the cities that have newer, better designed originally, and better maintained distribution plants. The New York supply has always operated under a comparatively low pressure, and this, again, has led directly to methods of construction that make for early and frequent defects in mains and service pipes. All this has a direct bearing on the probable amount in gallons per capita that now escapes from the New York distribution system without a useful purpose and the amount to which this waste may be reduced. In the United States the examples are yet few of special work done or much care taken to keep wastage within close limits. Those who have done such work find that the results accomplished are greater than one would suppose possible.

A few further notes: On p. 107, line 20 from the bottom, three-fourths should be changed to one-fourth. Page 108: It may be questioned whether the conduit rating method of measuring the flow of the several Croton aqueducts is the "only one practicable," and whether it gives "as accurate a measure of the discharge as any method of which we have knowledge to-day." On the filtration works built by the United States Government at Washington is a 6-ft. Venturi meter, built in place, of concrete, which registers flow accurately, day and night, the same as a common house meter. The writer twenty years ago described such a one built of wood inside of an existing 9-ft. pipe. There is nothing to prevent the construction of such an accurate meter to meter the flow of either of the Croton aqueducts any day, now or in the future, as part of the system of keeping a daily account of water income and outgo, which is necessary for the control of leakage and wastage; a control which Mr. Fuertes has admirably written to justify and recommend.

Clemens Herschel.

2 Wall Street, New York City, Aug. 10, 1906.

[Mr. Fuertes reply to the foregoing letter is appended.—Ed.]

Sir: Mr. Herschel politely pleads the never-ending struggle for perfection as a reason for writing the above interesting communication, containing suggestions for improvements, which, he states, with more time at the author's disposal might have occurred to him in writing the report referred to. Whatever Mr. Herschel writes is interesting and needs no excuse; for whether one agrees with him or not it will be conceded that his writings always contain food for thought, and that there are few men in the profession who write so easily and so gracefully.

The writer is indebted to Mr. Herschel for the trouble he has taken to point out, what, to him, seem to be faults in the report; apparently there is a misunderstanding of the distinction between the terms "leakage and wastage" and "unaccounted for water," as used in the report in question. The writer is therefore glad to have the opportunity to point out this difference with more emphasis. As a matter of fact, the writer can find very little difference between Mr. Herschel's convictions and his own on most of the points raised, and such as apparently do exist are more imaginary than real, when allowances are made for the distinction between "leakage and wastage" and "unaccounted for water" in metered cities, these terms being interpreted in the special sense in which they are used in the report.

The principal faults which Mr. Herschel is pleased to find are: (1) That the author assumes and expresses leakage and wastage as proportional to total consumption, and therefore properly expressed on a percentage basis.

(2) That "the further division of the 25% of the supply, of leakage and wastage, into a 15% that is 'effectively and quickly controllable,' and into a 10% that is 'beyond redemption,' is only an arbitrary one."

(3) That there is a "flaw in the author's argument, greatly vitiating the conclusions drawn in the report" from Table 12.

(4) That on "page 107, line 20 from the bottom, three-fourths should be changed to one-fourth."

Nowhere in the report is there a general statement that leakage and wastage are proportional to total consumption. The writer does not hold this view, and has studiously refrained from making such a statement, or from using data based on percentages for making comparisons between different cities.

When applied to any special or particular city, however, leakage and wastage may be expressed as correctly in percentages as in gallons per capita, or gallons per mile of main and service pipes. It would be equally improper to make a general statement that these losses in different cities are proportional to the miles of mains or to the per capita consumption. Cities cannot properly be compared or be placed in classes on either of these bases; nor does the writer see a reason why an attempt should be made to so classify them.

There is a difference between "leakage and wastage" and "unaccounted for water" in fully metered cities, as the terms are used in the report, which, apparently, Mr. Herschel has not kept in mind, for he quotes from Table 12, that leakage has been kept down as low as 8 gals. per inhabitant per day in American cities, and 1 gal. per inhabitant per day in German cities. Unfortunately, Table 12 contains no data on leakage; it shows only the estimated amount of unaccounted for water after leakage has been suppressed by universal metering. And this unaccounted for water may, in such cities, with propriety be expressed in percentage of total supply, consisting, as it does, largely of uncertainties in measurement, due to slippage of pumps, under-registration of meters, etc., which uncertainties are to a large extent proportional to total supply.

Mr. Herschel believes it possible to keep the leakage and wastage down to about 5 gals. per capita per day in New York; the writer believes this also, but he will not concede that the unaccounted for water, if universal metering were in force, could be kept down to anything like this low figure.

The writer has collected considerable statistical matter relating to under-registration of meters, new, and of various ages, on all kinds of services, steady and intermittent, on leaks, small streams, service conditions and full streams, for meters from ⅝-in. to 6-in., and he cannot bring himself to believe that in a city like New York the under-registration of consumers' meters would average less than from 8 to 10% of the water actually passing through them. This, alone, in New York, would be responsible for over twice the amount fixed upon by Mr. Herschel as the desirable and probably possible limit, assuming that Mr. Herschel includes the unaccounted for water, after universal metering has been accomplished, in the 5 gals. per capita.

The writer wishes to emphasize that Table 12 contains only cities completely metered, or nearly so, and in which, consequently, leakage and wastage have been reduced to the lowest amounts to which they can be held without an expense greater than would be warranted by the quantity of water still savable. The column headed "Unaccounted for water," therefore, shows not the amount of leakage and wastage still preventable, but the margin necessary in each case to cover under-registration of meters, slip of pumps in pumped supplies, errors in estimates of total supply in gravity supplies, undiscovered small leaks in mains and service pipes, small leaks at fixtures, uncontrollable wastage, and other losses always accompanying a complicated and extensive distribution system; and it is impossible, in any particular case, to separate this unaccounted for water definitely into its actual component parts.

Mr. Herschel's reference to the pipe-line of the East Jersey Water Co. is interesting, and the data valuable; but such data cannot be used as a guide in estimating the probable amount of unaccounted for water in a city.

Nowhere in the report referred to is, as alleged, the "25% of the supply, of leakage and wastage," divided into two parts, "a 15% effectively and quickly controllable" and a "10% that is beyond redemption." At least the author cannot see now why he should have made such a division, and he fails to find it in the report.

The author states that in his judgment, founded upon some study of local conditions, about 15% of the present total supply to Manhattan and the Bronx is lost by preventable leakage, and that this leakage could be quickly and effectively stopped by universal metering. The 25% does not refer to preventable leakage and wastage, but to the unaccounted for water after universal metering is in effect. He does not consider 25% "impossible of recovery for all time," but he does consider that, even with universal metering and with all preventable leakage and wastage stopped, the city will probably not be able to account, by means of meters on services, for more than three-fourths of the water entering the city's mains; very little of this could be classed as leakage and wastage.

No conclusions are drawn in the report from Table 12; therefore no flaw can exist in the author's argument greatly vitiating such conclusions. The data are merely presented as an interesting exhibit, almost without comment, for what they may be worth; and they may be useful or the reverse, according to the judgment displayed in their interpretation or application.

Regarding the suggested change on page 107, the paragraph in question reading:

> If all the water coming to the city were measured daily by the most accurate practicable means just before reaching the distribution system, and all water used for public, private, manufacturing and commercial purposes were sold by meter measurement, it would still be impossible to account, year in and year out, for more than three-fourths of the total supply.

Mr. Herschel says that "three-fourths" should be changed to "one-fourth." It is too late to make this change, and besides, it would be wrong. The writer, however, sees Mr. Herschel's point, and agrees that the English would be improved by substituting the word "one-fourth" for the words "more than three-fourths."

The writer has no desire to dispute Mr. Herschel's criticism regarding the remarks made in the report, on

page 108, concerning the gaging of the Croton aqueducts, which remarks were qualified, however, by the phrase "under the conditions," the author having in mind the probable difficulty of installing Venturi meters without interrupting the supply of water to the city. It could be done, easily enough, no doubt. There are very few things now classed as impossible among engineering undertakings, and, the Venturi meter, developed and made a practical device by Mr. Herschel, has proven its usefulness and reliability, for situations where it is applicable, in numerous places in addition to those mentioned by Mr. Herschel.

For the benefit of those who have not seen the report which Mr. Herschel discusses above it may be well to state briefly a general outline of the principal and more important findings.

About 15% of the total amount of water supplied to Manhattan and the Bronx Boroughs of the City of New York is lost by preventable leakage and wastage, a large proportion of which is from the fixtures of domestic consumers. With universal metering this 15% could be saved for use, thereby avoiding the probability of a shortage of water during the years that must ensue before the new works in the Catskill Mountains can be made ready for use. The leakage from the street mains is relatively small in amount and could not be stopped short of many years of costly work.

Manhattan and the Bronx are to-day supplied with more water than can safely be counted on from the Croton and Bronx watersheds, and a year or two of unfavorably distributed rainfall would bring about a serious shortage of water: possibly more serious than has occurred in the past. No amount of increased storage in the Croton and Bronx watersheds by flash-boards or new dams would add as much water of satisfactory quality to the average daily supply as can be saved from the present supply by universal metering. With all the precautions that can be taken to husband the present supply, by universal metering and inspections to locate and stop street main leakage, with all the additional storage that can be made ready in the Croton and Bronx watersheds, Manhattan and the Bronx will not be out of danger of a water famine until the new supply from the Catskill Mts. is made available.

The flow of the Croton River, on which Manhattan and the Bronx principally depend for water, has been above the average for several years, and the available supply has been correspondingly copious. The flow of the river, therefore, must soon fall below the average, and the available supply be correspondingly reduced, if the indications of the past 38 years can be used as a guide. Every effort must be made, therefore, to stop leakage and wastage as quickly and as effectively as possible.

The only significant amount of leakage that can be controlled quickly in Manhattan and the Bronx is that which takes place from the fixtures of domestic consumers, which can largely be stopped by systematic house-to-house inspections; but such an expedient would be extremely burdensome and extremely annoying to householders, most of whom are tenants, and not responsible for the conditions which would make this annoyance necessary.

The preventable leakage is the outgrowth of the carelessness of landlords in neglecting to keep plumbing and fixtures in repair. Leakage controlled by inspections will not be permanently suppressed, but will be resumed as soon as inspections are stopped and the money to pay for them, which would come out of the citizens' pockets, ultimately, would have been, in effect, wasted on a temporary expedient.

Universal metering will permanently control leakage and wastage; and with water rates properly adjusted would prove a more equitable and more rational basis for charges than the schedule rates now in force for domestic consumers. The use of meters would not restrict or curtail in any way the quantity used by domestic consumers, if a minimum charge be adopted sufficient to cover the value of as much water as is now used; but it will put a stop to the useless leakage by which, at the present time, upwards of 40,000,000 gals. of good water is allowed to run into the sewers every day, without doing anybody any good; and this 40,000,000 gals. per day, if saved, would be sufficient to supply over 400,000 people per day with all the water needed for every purpose.

The adoption of universal metering would not, when looking into the future, make necessary an increase of taxation; but it would lighten the cost of the new works to the city by making it possible to postpone the construction of the second large installment of the supply for from 12 to 15 years, the net result of which would be, after having fully paid for the meters and the maintenance of the meter bureau, to have saved in interest and operating expenses on the new works a sum amounting to from one-half to two-thirds the cost of the new works when they finally have to be built.

There is no possible justification, on any grounds, for the postponement of the construction of the first installment of the new supply, that is, of the supply from Ashokan Reservoir. This supply cannot be made available before the additional water thus secured will have been urgently needed, even with universal metering to control fixture leakage, inspections to find and stop leakage in street mains, and increasing the Croton supply by building the additional reservoirs now decided upon.

The conditions demand, then, that the new Ashokan Reservoir and the aqueduct leading therefrom be started immediately and be pushed to completion as rapidly as possible, and that to tide over the years that must elapse before this water can be made available, the fixture leakage in the premises of domestic consumers must be stopped as completely as possible.

As universal metering is the simplest, least annoying and most efficient means of securing the desired results in the control of wastage, and, as for Manhattan and the Bronx at the present time, and under the present conditions, the most economical for both tenants and landlords, as well as the most equitable, there should be secured to the city the right, which it does not now possess, to place meters on the services of domestic consumers, and to sell water only on the basis of meter measurement.

It may be well to point out explicitly at this point that the statements made above refer only to conditions in Manhattan and the Bronx Boroughs of New York City; no general deductions should be made therefrom for application to other localities.

Yours truly,

James H. Fuertes.

140 Nassau St., New York City, Aug. 16, 1906.

RAILWAY EXPANSION IN CANADA.

By J. A. Macdonald,* C. E.

More miles of railway will be built in Canada this year than in any previous year of her history.

A trunk line built through a country is but the base line of hundreds of lines of branches radiating in every direction from the trunk. For example, the Grand Trunk Pacific, whose road is now under construction in Canada, has obtained charters for some twenty-five branches (of fifty contemplated) which, when completed, will equal if not exceed the trunk line in mileage. Most of the construction work now going on, or soon to be undertaken, in Canada is trunk line, with the exception of the Canadian Pacific work, which is branch lines and extensions. So that, considering the class of work, I am safe in saying that no country in the world is carrying railway work, both actual and projected, to the extent that Canada is doing.

GRAND TRUNK PACIFIC.—This road (see Eng. News, Feb. 1, 1906) is engaged in the construction of the following lines: Main line, Winnipeg to Edmonton, 750 miles; branch to Fort William (which, strictly speaking, is a trunk line or rather a composite part of the Grand Trunk Pacific where it leaves the National Transcontinental Railway at Superior Junction), 220 miles. This line when completed will furnish direct line from Edmonton to Fort William, on Lake Superior, and will be open for traffic before the harvest of 1907. On reaching Lake Superior the wheat will be carried to points like Depot Harbor and Midland, on the shores of the Georgian Bay, where it will be transferred to the Grand Trunk Railway and conveyed to the ocean-going vessels at Montreal.

On the Government portion of the Grand Trunk Pacific (known as the National Transcontinental) from Winnipeg to Moncton, by way of Quebec city, contracts to the total amount of $20,000,000 have been let on the section from Quebec to LaTuque, 150 miles; and from Winnipeg to Superior Junction, 225 miles. Tenders will soon be invited for the construction of the Quebec-Moncton portion.

When the Temiscamingue Railway, owned and operated by the Ontario Government, is extended next year up to the Grand Trunk Pacific line, at the north end of Lake Abitibi, tenders will be asked for the construction of 100 miles of Grand Trunk Pacific from that point eastward. It is not probable that construction work will commence in the Quebec-Moncton portion of the line this year, as the tenders are not yet called for. Adding that mileage to the mileage being built by the company between Winnipeg and Edmonton, and between Superior Junction and Fort William, we have a total of about 1,350 miles belonging to the new Transcontinental Railway now under construction.

CANADIAN PACIFIC.—This road will build 900 miles of new line, chiefly in the West this year, and in addition it is pushing the double-tracking of its main line between Winnipeg and Fort William, a work that will cost $5,000,000.

CANADIAN NORTHERN.—This railway will build a branch 90 miles long, from a point on the Prince Albert line into the Carrot River country, a famous wheat region. Next year this branch will be carried towards Fort Churchill on the west coast of the Hudson Bay. Two extensions will be built from Edmonton, one running north to Athabasca Landing, 80 miles distant, the other going west to the Rocky Mountains. The company is also continuing the construction of its line from Toronto, passing Lake Simcoe and crossing the main line of the Canadian Pacific, towards James Bay, besides doing considerable new work in Nova Scotia.

GREAT NORTHERN.—The plans of the Great Northern, Mr. J. J. Hill's road, for this year do not appear to have been finally determined, but the road along the International boundary to Vancouver will be pushed, and lines from Gretna to Portage Laprairie, and from Bottineau to Brandon, as well as a spur into Winnipeg, are now under construction.

SANITATION.—The Dominion has employed two inspectors under the Public Works Health Act to watch and report on the sanitary conditions of the railway camps wherever 100 men or more are employed. Dr. Chamberlain, formerly inspector of prisons for Ontario, is the inspector between Winnipeg and the Pacific Coast. Another, probably Dr. Fisher, will look after the camps between Winnipeg and Atlantic.

LABOR PROBLEM.—The labor problem is perhaps the most serious that will have to be faced by the various contractors for railway work in Canada this year.

The work here enumerated will probably afford employment, within the next few months, for some 20,000 men, and as the work will continue from year to year for seven or eight years at least, this number of men will be constantly employed for that period.

PROJECTS.—The Canadian Northern (MacKenzie & Mann) has obtained charters for a line from Key Inlet, on the Georgian Bay, to Montreal, via Ottawa, and from Toronto to Ottawa, for some 600 miles of new line. Construction work on the Ottawa-Montreal portion is now going on.

It is said that Mr. Hill, of the Great Northern, has practically decided to build a trunk line from Winnipeg to the coast, as well as another shorter line from Vancouver by way of Fernie, making a junction with his main line at or near Medicine Hat, and which he promises to have completed simultaneously with the completion of the Grand Trunk Pacific.

The Grand Trunk Pacific has obtained authority to construct branches from the main line to Prince Albert, Regina, Calgary and Brandon, while the Government-built portion, from Winnipeg to Moncton, has branches outlined to Montreal, Ottawa and North Bay.

In addition to the above, the Federal Parliament, during the last days of the session, voted the sum of $9,320,000 in subsidies ranging from $3,200 up to $15,000 per mile toward the building of 43 branches, covering 2,885 miles of railway in the various provinces of the Dominion.

The mere statement of these facts is enough to convey some idea of the rapidity with which the country is developing, through the increasing application of labor and capital to the exploitation of its natural resources. It should be borne in mind that none of the enterprises thus financed are of a speculative character. The region through which the new railway lines are to be constructed are well known, and the great corporations which have undertaken to furnish them with transportation facilities are taking no chances of failure. The great increase in such facilities is made because it is urgently needed, and, so far as at present appears, the work will have to be kept

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in progress for many years to come. The natural resources of the country are vast, varied and widely distributed, and the result of this development will be an incalculably great impetus to the prosperity and solidarity of the Dominion.

FOREIGN RAILWAY CONSTRUCTION IN SLIDING GROUND.*

By E. E. R. Tratman.†

Railway engineers in Europe have had very extensive experience with construction and maintenance in treacherous soil and in some cases very elaborate and costly works have been necessary. Many of these works have been necessitated by and built under peculiar and exceptional conditions, and it is of interest to put on record some of the difficulties encountered and the methods adopted for overcoming them.

PARIS, LYONS & MEDITERRANEAN RY.

The Paris & Montereau division of the line from Paris to Lyons, France, was opened in 1849, and it was stated in a paper written several years ago by Mr. Cartault, one of the division engineers of that road, that for some forty years there were movements of the ground, sometimes to such an extent as to block the road. It was only by very thorough investigations as to the causes, and the execution of very extensive works, that stability was finally assured. The country traversed by the railway has a surface of loam and alternate permeable beds of limestone or sand and impermeable beds of clay, all underlaid by compact chalk. The beds are very regular and have not been subject to upheaval since their formation. The rainwater soaking into the loam flows along the first bed of clay; that and the lower beds are mainly dry, except where water is given access to them by the railway or other works. It would seem that cuts in such a formation would not disturb the equilibrium of the mass, as the surface water flows over the slopes and into the roadway ditches; the lower beds would then remain dry and stable, as the clay is only dangerous when made wet. This did not prove to be the case, however, and in a certain length of time, sometimes several years, certain slopes began to slide. This is explained by the engineers as follows: The opening of the excavation cuts the permeable beds alternated with the clay beds. The water flowing on the slopes or in the ditches wets the edges of these beds. If the beds are inclined towards the railway the water can only work between them by capillary action, and causes little trouble, as it does not extend to any distance, but when the slope is in the opposite direction the water gradually penetrates to a considerable distance. The slope then rests on an unstable bed, and a subsidence occurs which is increased by the vibration due to the trains, and a crack or crevice opens along the slope, into which the rainwater enters. Finally a mass of greater or less size detaches itself, and moving against the direction of the slope of the bed falls into the cut.

One of the most troublesome cuts was that at Mee, with a maximum depth of only 23 ft.; the slopes are 45°, with a toe wall of dry masonry on each side, and the ballast supported by dwarf walls to form a ditch. This is shown in Fig. 1. The cut yields a constant and considerable flow of water, even in dry seasons. Beneath the vegetable soil is a permeable bed of limestone, then a bed of permeable clay, a second bed of limestone, a bed of sand, and finally another bed of clay on the rock. The inclination of the beds is generally opposite to that of the surface of the ground. The water flows entirely from one side of the cut, and this side has never slipped. The slides occur on the opposite side, which shows no trace of moisture. Where the bed of limestone and sand is below the roadbed, it receives no water from the excavation and no slides occur; and where the bed comes behind the dwarf retaining wall, the latter protects it, and no slides occur. But where the sliding plane is above the wall, the water of the slope gradually works along it and causes slides. Dry retaining walls carried below the ditch were of little effect, as they simply admitted water to the treacherous bed. After the causes had been thoroughly investigated, a remedy was effected by simply deepening the opposite ditch and diverting all the water on to that side. In some cases a stone wall was built to cover the edge of the permeable bed, and so prevent the water from flowing into it, as shown.

The bank following this cut is in some cases 60 ft. high, and was protected by a drain formed by a deep trench filled with dry stone. But it became necessary later to build a masonry drain or gallery 5 ft. high and 2 ft. 4 ins. wide below the level of this wall. The gallery is 840 ft. long, at right angles to the railway. The head of the gallery is directly beneath the bank.

Near Senart is an arched masonry culvert which was broken into four sections by cracks parallel with the rails, due to the sliding of the beds on which the foundations were built. A masonry drain was built under the foundations, with branches forming a Y and terminating at vertical wells opening into the ditch, thus draining the lower beds and also carrying away the surface water which had formerly penetrated the earth. In a number of other cases extensive drainage galleries were built to protect bridges where slides occurred and embankments, which in some cases had sunk some 10 ft. below the original level, the track having been raised from time to time. Fig. 2 shows the main gallery carrying the water from a system of drains in the hillside above the railway. Fig. 3 shows the methods adopted at the bank approaching the bridge at Brolles, which had begun to slide. A trench 6.5 ft. wide was built on the upper side, extending through the permeable strata, so as to intercept the flow of water. At the bottom of this was built a masonry gallery or drain, backed by concrete on the downhill side, and otherwise covered by rubble stone filling. Openings in the roof and up-hill wall admitted the water. The drainage gallery led to a transverse gallery or culvert passing under the bank and discharging the water far below it down the hillside.

LONS-LE-SAUNIER & CHAMPAGNOLE RY.

Another case of extensive and almost hopeless conflict with sliding ground was on the railway from Lons-le-Saunier to Champagnole, in France. As described by Mr. Meron, the chief engineer, some 14 years ago, the

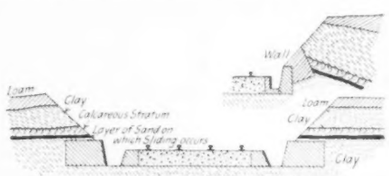


Fig. 1.

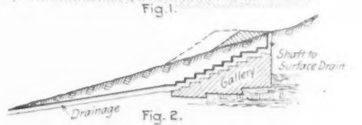


Fig. 2.

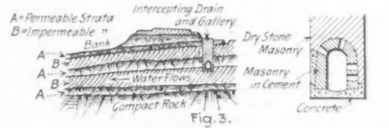


Fig. 3.

Fig. 4.

Fig. 5.

railway rises from Lons-le-Saunier by a grade of about 2½% to reach the crest of the first plateau of the Jura range. It cuts successively all the beds of liassic formation, which are exceptionally unstable in that district, and the situation is aggravated by the transverse slope of the hill on which the line runs, and by the presence of numerous springs. The grade has a length of about seven miles. Work was commenced in 1881, and as no provision had been made for the drainage of the land, the work on the heavy cuts at once disturbed the equilibrium of the strata, and as fast as the movement was stopped in one place it developed at another. The banks also subsided, and in the winter of 1882-83 the ground slid into the Vertancul cut for the entire length. In November, 1884, the hill above the village of Perrigny moved bodily and completely closed the Vieux-Mont cut for a length of 150 ft. This slide was repeated on a larger scale two years later. At the opening of 1885, the three miles of line built showed nothing but settling banks and sliding cuts. Extensive drains were built around the works, and transverse trenches filled with broken stone; but the drains only worked for a time, while the stone of the ditches settled into the ground, and in a year or two the movements recommenced. The contractors threw up the work, and the engineers, after long and careful investigations, prepared designs for the drainage and consolidation of the land. These were approved in 1886 and the works were completed in 1888. This was done without deviation from the original line, except at the Vieux-Mont cut already mentioned, where the consolidation would have cost some $80,000 on a length of 1,300 ft., as it would have been necessary to sustain a hillside of some 60 acres, moving as one mass with a depth of from 33 to 50 ft. At this point the old cut was abandoned, and the line on the new location was thrown further out and carried over a small valley or ravine by a system of drainage galleries. This bridge naturally spanned a slide. In December, 1886, another slide occurred on an area of 60 acres, and flowed into the old cut, but fortunately the ground between the old and new lines resisted the pressure. This section of the road, 4¾ miles long, had been estimated to cost $318,220, but it actually cost $650,000, or about $136,000 per mile.

THE LIEME EMBANKMENT.—This bank is located at the beginning of the line, on approximately horizontal ground, and has a maximum height of 41.5 ft. In view of the character of the material necessary to be used, stone mounds the entire width of the bank and about 12 ft. high were built on the ground at intervals of 130 ft. before the material of the bank was deposited. This clay was very wet, and the bank soon settled and slipped for a length of 650 ft., but without disturbing the stone drains or mounds. In order to maintain the line open for traffic, new material was deposited on the top and also on the sides to load it until the bank was in some places 200 ft. wide at the base. Fig. 4 is a section at one of these mounds. Trenches five ft. wide, and with vertical sides, were then made across the flattened slopes of the bank and filled with stone to the level of the old drains, reducing the intervals between drains to about 43 ft. Both old and new drains were extended at the ends to conform to the actual slope of the bank, as shown in Fig. 5. Headings were then driven through the bank to connect the opposite lines of stone drains, the headings or galleries being filled with dry stone masonry, leaving a box drain at the bottom. These drifts showed that while the soil of the bank had been largely dried out, there were veritable pockets of mud and water, so that in places the distance between cross drains was still further reduced to 21.5 ft. The base of the bank having been thus treated, the slopes of the upper part were cut in horizontal benches and new slopes filled in with good material brought from a distance; this was sown with lucerne and rye grass to form a heavy turf. It might seem that it would have been cheaper to build the entire bank of good material brought from a distance, but it is estimated that this would have cost 70% more than the works described.

THE PERRIGNY CUT.—This is a long cutting, directly following the Lieme bank above mentioned; it is 2,500 ft. in length, but the trouble was mainly for about 1,200 ft. at the middle, where the maximum depth was about 50 ft. The cut was first started by excavations to full depth, but this at once started sliding and caving of the ground, and it was found that the underlying material through which the lower part of the cut lay was not stable, as had been supposed. It therefore became necessary to excavate the prism bounded by the surface of the ground, the left slope, and the horizontal plane level with the top of the right slope. This exposed a limestone and chalk formation, in which the final cut was made with slopes of 45°, instead of 1¼ to 1 as had been intended. A section of the cut is shown in Fig. 6. This latter slope was used in the upper part of the cut, but successive slides reduced this to 1 on 2 in the upper slope on the left side: transverse trenches were cut 33 ft. apart and 6 ft. wide, the bottom being benched and having a slope of 1%; these were filled with stone, leaving a box drain at the bottom. The first of these trenches was not carried down to the good rock, but this proved to be unsatisfactory, and all others were thus deepened. During the excavation of some of these trenches the wet ground slid and was so soft that the trench timbering would not hold but was distorted. In spite of the severe cold weather a retaining wall of masonry in mortar was built at the foot of the slope; this was moved somewhat by the pressure, but served as a bearing for the timbering so that the trenches were at last made; these cross drains dry out the material, which would otherwise not be held back even by the heavy wall. A masonry drainage ditch was also constructed along the entire length of the top of the slope on the left side. Finally the slopes of the cut were lined with a dry-stone revetment, to protect them from the disintegrating action of the air, and acacias were planted on the upper slopes to consolidate the ground. The total cost of this one cut was about $116,500.

WORKS AT CONLIEGE.—In approaching the town of Conliege the line passes into a very unstable stratum which gave almost endless trouble. It is underlaid by schist which provides an excellent support for foundations, but unfortunately there is a bed of soft clay, full of springs, between this and the surface clay. The water from the springs saturates the surface clay to such an extent that it will not carry any load, and all the proposed embankments have had to be abandoned, after work upon one had started a slide of the whole hill for a length of 700 ft.; it was finally stopped by a retaining wall carried down into the solid substratum. A drainage project was prepared by the engineers, approved by the government in 1883, and after much difficulty completed in 1885, when filling was again commenced on

*Abstract of a paper read before the Western Society of Engineers, and reprinted from the June number of the Society's "Journal." The same number of the "Journal" also contains a short paper on the slide at the Boone viaduct of the Chicago & Nortwestern Ry.; the slide was checked by making stone-filled trenches to drain the wet ground.

†Associate Editor of Engineering News, Chicago.

the bank. Two days of rain, however, started a new movement, and in one night the bank slid to the foot of the hill, pushing the soft soil below it like a wave. Still a third bank, only eight feet high, behaved in the same way, and it was evident that even the costly drainage works could not make the surface stratum stable.

It was therefore necessary to build masonry viaducts, with piers carried down to the solid substratum. The piers are 39.3 ft. between centers, with semi-circular arches of 16.4 ft. span. The Vertancul viaduct has five arches and the Ermitage viaduct, Fig. 7, has six arches. The excavations for the piers were carefully filled with masonry, so as not to leave voids, which might start slides. It became evident, however, that it was impossible to prevent the formation of a sheet of water around the foundations, which would gradually soften the earth until a slide would occur that might carry away the piers. All the piers, therefore, had on each side a dry stone wall two feet thick, carried below the base of the foundation and having at the bottom a box drain; the two drains of each span connected with a Y-shaped drain, the stem of which led to a general drainage system. One of the viaducts is set with its upper side close against the hill, no parapet being used on this side, with a paved ditch outside the ballast. Beyond the ditch is a six-foot trench carried down to the good substratum and filled with dry stone, a box drain at the bottom connecting with the general drainage system. The station grounds are formed by a side-hill cut, and drains consisting of trenches filled with stone are placed in the up arch down the slope, while A-shaped cross drains between them intercept the surface water and divert it into the main drains.

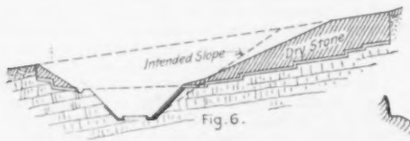
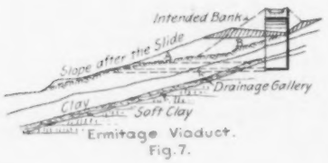


Fig. 6.



Ermitage Viaduct.
Fig. 7.

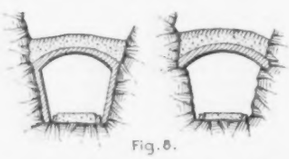


Fig. 8.

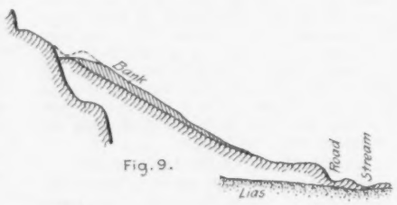


Fig. 9.

Fig. 10.

Fig. 11.

COVERED CUTS.—In passing through rocky districts, some of the long side-hill cuts with steep slopes 65 to 100 ft. high, were subject to serious falls from above, due to the action of frost and ice. This was checked in some cases by a stone revetment of the steep slopes, but in two cases it was found more economical to build a masonry roof over the cuts. These are shown in Fig. 8. The arched roof is in one case seated in the rock of the slopes, while in the other cases it is supported by inclined side walls built against the slope of the original cut, the rock being too badly disintegrated to afford support. A bed of filling above the roof protects it from falling rocks.

THE ROCHECHIEN DIVERSION.—In the original location the line was intended to pass around the rocky spur overlooking the village of Revigny by a side-hill bank, beginning at the mouth of a short tunnel. The slope of the surface was about 1½ to 1, and at the foot was a government road and the Rochechien Creek. The top of the bank was very narrow, but owing to the slope of the hill it extended far down the slope, and at one place a masonry arch bridge of 43 ft. span was built across a depression. In a few months the arch was found to be cracked and the abutments moving while crevasses formed a great A in the face of the hill, the apex being just above the bank. The bank moved about three feet out of line, and sank about two feet. Three deep exploration shafts were sunk to determine the actual geological conditions, and it was found that the underlying rock had a steep slope and was covered with a bed of moist clay so that there was no hope of making the bank stable. This is shown by the section, Fig. 9.

The new plan adopted was to throw the line back into the hill so as to lie in the rock, crossing the ravine filled with the treacherous material by a truss bridge supported on the rocky sides. The line was therefore continued in tunnel right to this bridge; part of this tunnel coincided with a great longitudinal fault in the rock, and one side wall had to be supported on an arch, while cross walls or buttresses were built from the side wall to the face of the rock. This is shown in Fig. 10. Beyond the bridge, the line was built partly in a masonry gallery along a side-hill cut, the gallery resembling a tunnel (but built in the open) and having arched openings in the down-hill side; Fig. 11. This diversion or deviation line cost about $54,000.

THE ALBULA RAILWAY.

On the Albula Railway, in Switzerland, very heavy mountain works were required, and many tunnels and masonry viaducts (the latter with piers carried to considerable depths) were built in preference to open cuts in steep side-hill ground where heavy retaining walls would have been required. Much of the work was in loose material overlooking steep rock slopes and liable to slides. Open cuts often exposed strata of wet clayey material, very unstable, and in some cases the pressure caused this to slip, with the result that the entire superincumbent mass went with it. In most cases, rapid timbering of the style shown in Fig 12 enabled the movement to be checked temporarily while very heavy retaining walls were built, as shown in Fig. 13. In one case on the Cugnelet Division, an extremely heavy wall with a face batter of ⅛ to 1 was used, having courses laid perpendicular to the face so as to aid the resistance to sliding under the pressure from behind. The wall was not carried the full height of the cut, but only high enough to hold back the treacherous stratum, while the weight of the upper stratum of good material rested on top of the wall.

Even tunnels did not escape, however, and in the Versasca tunnel a very heavy concrete floor had to be built to prevent the walls from closing in. This floor was flat on top, and the bottom was formed in a curve so as to give great thickness at the middle. When the cut at the tunnel portal began to slide, timbering was put in and held it until a heavy retaining wall could be built.

MISCELLANEOUS CASES.

The relations of geological conditions to certain forms of slides are described in a recent paper on "Geology in Relation to Engineering" in "The Engineer," of London. Fig. 14 illustrates a case where a railway embankment on side-hill ground was undermined by the surface water flowing down the hill side, so that the bank slipped bodily, and at the same time spread out at the base. This was remedied by cutting a ditch parallel with the upper toe of the bank and then tunneling through the bank for culverts to carry the water to the lower side. It is stated that with the strata shown the bank would hardly have been expected to give away, as a large portion of the water would naturally soak away into the ground, while the probability of a collapse would have been greater if the surface had been of a rocky nature. It is most likely, however, that the loam became so saturated as to be unstable and unable to carry the load due to the bank, as in the case of the French railway noted above.

Cuts which are apt to be very troublesome are those passing through clays which are apparently very stiff when dry (or nearly so), and will stand at slopes of 1 to 1, or 1½ to 1, when freshly excavated, but which turn almost to mud after being exposed to the air and weather for some time, and will then only be stable at slopes of 3, 4 or 6 to 1. The only alternatives are (1) to use a flatter slope (which may require the purchase of more land); (2) to excavate a large portion of the slope, burn it like clay for ballast, and then replace it; or (3) to drain the slope by cutting trenches in the face and filling them with stone, afterwards forming the cut to its original or a flatter slope, if possible. The way in which the sides of such a cut will slip is shown in Fig. 15.

An instance of a slip in a cut on a German railway is shown in Fig. 16, and shows the action of a slide against the inclination of the strata, as in a case on the Paris, Lyons & Mediterranean Ry., already described. On this German line the cut was through side-hill ground with alternate layers of sandy gravel and clay. The surface drainage after heavy rains passed down the slope on the upper side and into the track ditch without doing any damage. On the lower side, however, it percolated through the strata until they were so saturated that masses would slide into the cut, as shown by the dotted lines. In some cases it required 40 years for this result to develop. Cuts through side-hill ground in such formation as the lias, which consists of alternate strata of rock and shale, etc., and which strata may dip at a steep angle, should be carefully watched for landslides; these are very liable to occur, especially after hard frosts, rain and snow. The water penetrates the fissures, and by its freezing and swelling disintegrates the shale; the result is that the rock, losing its support, and shaken by the vibration due to passing trains, breaks away and falls into the cut, as shown in Fig. 17. During the excavation of such a cut, the dip of the strata should be measured and plotted as a cross section, so as to show if trouble of this kind is likely to occur.

An extremely interesting paper on certain properties of clay as related to the occurrence of slides was presented before the Canadian Society of Civil Engineers in December,* 1902, by Mr. H. J. Cambie, and is worth noting here. It appears that clay which is perpetually moist will absorb only a certain further percentage of water, and will retain its form and consistency. But if dried out and then again subjected to the influence of water it will absorb a greater percentage than the normal, and will then turn to mud, having no cohesion. In a certain portion of British Columbia the rainfall is very light and the water penetrates only a few inches, being dried up by the rapid evaporation. The Canadian Pacific Ry. cuts in the dry clay were very stable, the clay being hard and resembling soft soapstone. But when irrigation was introduced in that country, landslides began to occur along the railway, and investigation showed that under

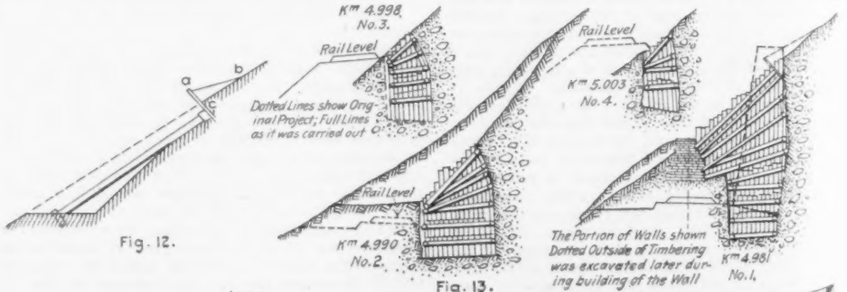


Fig. 12. Fig. 13.

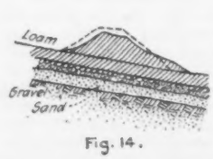


Fig. 14.

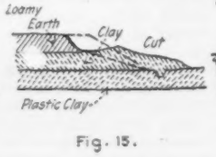


Fig. 15.

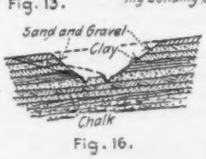


Fig. 16.

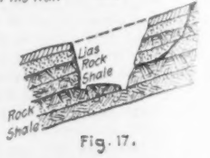


Fig. 17.

*Eng. News, Jan. 8, 1903; pages 34 and 38.

the influence of moisture the hard, dry clay lost its cohesion. In the tests, a block of the dry clay was placed upon a plate and water dropped upon it; it absorbed 50% of its own weight of water without any change of form or other visible effect, but when it had absorbed about 60% of water it completely collapsed and became as fluid as water. An English writer states that sandy clays, weighing 113 lbs. per cu. ft. when dry, will readily absorb water and turn into a sludge with an angle of repose not exceeding 10°. "Argillaceous silt will absorb 53.5% of its volume without altering its form, but after absorbing 78.5% it disintegrates and becomes a slurry. Blue lias clay will often not stand at slopes of 4 and 5 to 1, when saturated with water, and plastic clays at slopes of 7 to 1.

It has been suggested that tile may be economically used in land drainage work of the kind referred to, but it is evident that it would be of little use in such treacherous soil as that encountered by the French railways mentioned. A very slight movement or slip would dislocate and disconnect the drains, rendering them useless. With the stone-filled trenches, however, considerable distortion may take place without interrupting the continuity of the drain. On many European railways considerable trouble has been experienced from slides in wet cuts and in wet lands, and the use of drains formed of trenches filled with stone is very general. In all the cases mentioned, the aim has been to secure permanent stability and safety, and not merely a temporary relief.

THE STATE ENGINEER AND HIS RELATION TO IRRIGATION.

An interesting bulletin with the title given above has been issued by the U. S. Department of Agriculture. The detailed information which it contains is of more particular interest to engineers and others in the West; but the opening paragraphs, on the general subject of state engineers, and also some tabular summaries at the close of the bulletin, are of sufficient general interest to warrant quoting here as follows:

The office of state engineer is peculiar to the arid states, and the duties of the office have to do chiefly with irrigation. These duties range from merely making hydrographic surveys, as in Oregon, to practically complete control of the water supply, as in Nevada. It is true, however, that in most of the arid states whatever there is of public control of the use of water centers in the office of state engineer. A comparative study of the work of the engineers involves, therefore, the whole subject of public control of water.

The final purpose of the creation of the office of state engineer is the delivery of the water to which he is entitled to each farmer whose farm is under irrigation, and a necessary preliminary to this is a knowledge of how much each is entitled to. The early laws governing irrigation were not such as to provide a record of rights, and it is therefore necessary to provide means for defining existing rights. This in some states has been committed to the engineer, in others he makes surveys to assist the courts in this work, while in others he has nothing to do with it. To avoid future trouble, it is also necessary that hereafter rights be defined as they are acquired, and several states have provided that rights be acquired under the supervision of the engineer, while others still have no provision for supervising the acquirement of rights, but leave the defining of the rights to be done when controversy regarding them arises. The final act—the distribution of the water of streams to those entitled to it—is in some states all that devolves upon the engineer, while in some states the engineer has not this duty, but has a part in the proper defining of rights.

PUBLIC CONTROL OF WATER, 1905.

State or Territory.	Defining of Rights.	Acquirement of Rights: Initiation.	Proof of Completion.	Distribution.
Arizona	No provision	Post and file notice	No provision	No provision.
California	" "	" "	" "	" "
Colorado	Courts—on application of interested party	" "	" "	Public officials
Idaho	Courts—surveys by state engineer when ordered by court	Application for permit	Inspection by state engineer	" "
Kansas	Courts	Post and file notice	No provision	Court officers
Montana	Courts—all claimants parties; no surveys	" "	File maps	" "
Nebraska	Administrative—secretary, board of irrigation	Application for permit	Sworn statement and inspection	Public officials
Nevada	Administrative—state engineer	" "	Rules not made	" "
New Mexico	Administrative—board of control.	File notice	No provision	No provision.
North Dakota	Courts—surveys by state engineer	Application for permit	Inspection by state engineer	Public officials
Oklahoma	Courts—surveys by territorial engineer	" "	Inspection by territorial engineer.	" "
Oregon	Courts—surveys by state engineer when ordered by court	Post and file claim	No provision	No provision.
South Dakota	Courts—surveys by state engineer	Application for permit	Inspection by state engineer	Public officials
Utah	" "	" "	Sworn statements	" "
Texas	No provision	File notice	No provision	No provision
Washington	Courts—on application of any interested party	Post and file notice	" "	Public officials
Wyoming	Administrative—board of control.	Application for permit	Inspection by superintendent	" "

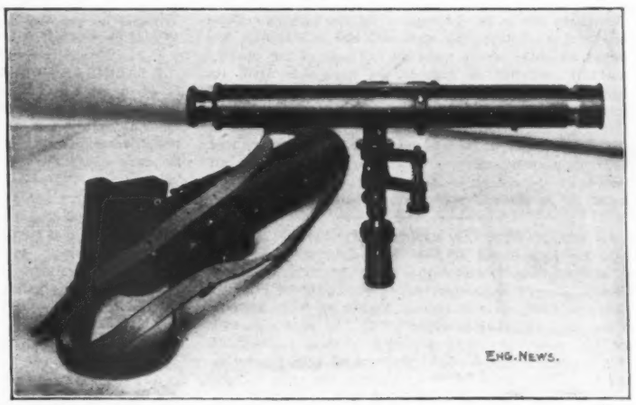
IRRIGATION OFFICIALS IN THE ARID STATES, 1905.

Officials.	Appointed by.	Term.	Salary.	Paid by.
Colorado:				
State engineer	Governor	2 years	$3,000 per year	State
Division engineers	"	"	$125 per month	"
Water commissioners	"	Not fixed	$5 per day	Counties
Idaho:				
State engineer	"	4 years	$2,000 per year	State
Water commissioners	"	6 years	$10 per day	State & counties
Water masters	Water commissioners	1 year	$4 per day	Users
Kansas:				
Water bailiffs	Courts	Not fixed	$2 per day	Counties
Montana:				
State engineer	Governor	4 years	$2,500 per year	State
Water commissioners	Courts	Indeterminate	Not fixed	Users
Nebraska:				
Secretary state board	State board	2 years	$2,000 per year	State
Under secretaries	" "	" "	$5 per day	"
Under assistants	" "	" "	" "	Counties
Nevada:				
State engineer	Governor	Not fixed	$2,400 per year	State
Water commissioners	State board	"	Not fixed	Counties
North Dakota:				
State engineer	Governor	4 years	$2,500 per year	State
Water commissioners	"	6 years	$10 per day	"
Water masters	Water commissioners	Indeterminate	$4 per day	Users
Oklahoma:				
Territorial engineer*	———	———	———	———
Water masters	Territorial engineer	———	$3 per day	Users
Oregon:				
State engineer	Governor	4 years	$2,400 per year	State
South Dakota:				
State engineer	"	6 years	$2,000 per year	"
Water commissioners	"	"	$5 per day	"
Utah:				
State engineer	"	4 years	$3,000 per year	"
Superintendents	State engineer	Indeterminate	Fixed by engineer	"
Supervisors	County Commissioners	"	$3 per day	Counties
Washington:				
Water commissioners	" "	1 year	Not fixed	"
Wyoming:				
State engineer	Governor	6 years	$2,500 per year	State
Superintendents	"	4 years	$1,200 per year	"
Water commissioners	"	2 years	$5 per day	Counties
Secretary board of control	Board of control	Indeterminate	$1,200 per year	State

*Duties prescribed but no provision for appointment.

AN IMPROVED HAND-LEVEL, WITH STADIA WIRES.

A hand-level fitted with stadia-wires, and otherwise developed for more accurate work than the long-standard type of Locke hand-level can do, is a novelty which has recently been put on



STADIA HAND-LEVEL, WITH SLOW-MOTION LEVELING ATTACHMENT AND QUICK-LEVELING BASE FOR JACOB-STAFF.
Keuffel & Esser Co., New York, N. Y., Makers.

the market by Keuffel & Esser Co., of New York City. The instrument, called the stadia hand-level, is shown in the accompanying photograph, as fitted for mounting on a Jacob-staff; when the two base sections are taken off the tube itself is complete for use "offhand," like the ordinary Locke instrument. The principal new features of this hand-level, as compared with the Locke, are: (1) stadia wires, for reading distance; (2) telescope, with achromatic objective, giving increased definition; (3) stadia and level lines of regular cross-hair construction; (4) Jacob-staff head, with ball-and-socket joint for quickly leveling up; (5) slow-motion leveling attachment, for setting the instrument accurately level in the line of sight. The last named, shown in place in the accompanying photograph, is an "extra"; all the others are regular elements of the stadia hand-level as sold. A leather case furnished with the instrument makes it very easy to carry about; in complete form, with slow-motion leveling attachment and case, it weighs only 29 oz.

The telescope, 10½ ins. in length over all, has a magnification of eight diameters. The objective, which is 1 in. in diameter, and eye-piece (erecting) focus independently; the cross-hairs and the bubble-tube are set so as to be focussed simultaneously by the eye-piece. The stadia interval is 1:100. The level bubble has a value of 0° 5′ per 0.1 in. of motion. The quick-leveling base comprises a head for the Jacob-staff, a ball joint, and a revolvable cap section which screws to the instrument tube; a set-screw is fitted to the cap section for holding the instrument from rotation when desired. The slow-motion leveling attachment, when used, screws into the instrument tube in place of the quick-leveling base, and receives the screw of the latter in its lower portion; it consists of an upper and a lower portion connected by a horizontal hinge and a vertical slow-motion screw, with opposed spring. This attachment, of course, serves only to level the instrument in a single line of sight.

As compared with the simple hand-level, this instrument, when used "offhand," enables distances to be obtained without direct measurement, and probably gives somewhat better accuracy of leveling. As thus used, without base, the instrument weighs just one pound. When used with a Jacob-staff or tripod it makes it possible to run fairly accurate lines of levels at

the cost of but a fraction of the weight and bulk of a regular level. These combined possibilities, and the low price ($18, without slow-motion leveling base), seem to make the instrument a very convenient tool for the engineer, in a large variety of uses.

A NATIONAL TEMPLE OF JUSTICE, or building for the U. S. Supreme Court, is propoced at Washington. It is stated that Mr. Elliott Woods, Superintendent of the Capitol, in conjunction with New York architects, will have the plans ready to submit to Congress in December.

BLOCK SIGNALING AND ELECTRICAL SIGNALING appliances are to be investigated by the Interstate Commerce Commission. To this end the commission has issued an order calling upon all railroads of the country for full statistics as regards the apparatus used, its operation, etc. The information thus received will be embodied in a report to Congress, and will show the merits of the block signal, the progress being made in the introduction of improvements, how generally used and the necessity for such signals. The value of all devices for the control of trains that have been adequately tested in service will also be investigated.

AN INTERNATIONAL MINING EXPOSITION is to be held for four weeks at New York City early in 1908. It will be open to the entire world for exhibits of all kinds of mining machinery, minerals, chemicals, precious stones of every nature, and all known kindred products of the earth, whether liquid or solid. The exposition will not be competitive regarding the different exhibits of minerals or machinery. Minerals from all mine properties will be eligible for exhibition, whether the mine is partially or fully developed, providing such properties submit and keep on exhibition for inspection a report, from a reputable mining engineer, stating the location and degree of development of the mine. Selling of mining stocks will be strictly prohibited, but exhibitors of precious stones or high-grade minerals may vend the same from the booth in form of jeweiry or souvenirs by obtaining a permit from the management. Each state will be asked to prepare an exhibit of such minerals as may be found within its confines. Exhibits illustrating mining camps of different nations, the manner of transporting ore over the mountains on the backs of burros, the evolution in mining machinery and tools, and their manipulation will be provided, as well as a complete working laboratory showing the extraction of all kinds of metals from their respective ores. The exposition will be under the direction of the International Mining Exposition Co., 564 Monadnock Bldg., Chicago, Ill., Mr. Wm. M. Porter being Director General.

GAS AND ELECTRIC LIGHT RATES AT SYRACUSE, N. Y., have been ordered reduced by the State Gas and Electric Light Commission, as a result of a petition submitted to it under the terms of the act creating the commission. The price of gas has been ordered reduced from $1 to 95 cts. per 1,000 ft., and of incandescent electric lighting from 12 cts. per KW. hr., with 2 cts. discount if paid within a few days of presentation of bill, to a rate of 9 cts., with a further reduction to 8 cts. after Oct. 1, 1907. The charges for arc street lamps have been ordered reduced from $85.77½ to $68.

LOCOMOTIVES FOR CHINA are being built by Robert Stephenson & Co., of Darlington, England, for the Imperial Chinese Railways, and include ten engines of the eight-wheel (4-4-0) type for service on the Shanghai and Nankin line, 200 miles. The engines have outside cylinders, Belpaire fire-boxes, and six-wheel tenders; and are equipped with the Westinghouse brake. A hood over the front end of the tender deck shelters the fireman and forms, with the engine cab, a complete enclosure. A bulkhead shuts off the coal space from the tender deck. The principal dimensions are as follows:

Driving wheels	6 ft. 7 ins.
Truck wheels	3 ft. 6 ins.
Wheelbase, driving	8 ft. 3 ins.
Wheelbase, engine	25 ft. 2 ins.
Wheelbase, engine and tender	48 ft. 9 ins.
Weight on drivers	83,100 lbs.
Weight of engine	126,000 lbs.
Cylinders (2)	18 × 26 ins.
Tubes, 188; diameter outside	2¼ ins.
Tubes, length	12 ft. 10½ ins.
Working steam pressure	180 lbs.
Heating surface; tubes	1,467 sq. ft.
Heating surface; total	1,630 sq. ft.
Grate area	28 sq. ft.
Water on tender	3,500 gallons
Coal on tender	7 tons.
Tractive power	16,315 lbs.

THE SEWAGE WORKS OF SHEFFIELD, ENGLAND, treated a total of 7,108,296,000 U. S. gals. in the year 1905-6. Of this amount, 376,440,000 gals. were treated by the bacteriological process, and the balance by chemical precipitation with lime. The average daily amount treated was 19,440,000 gals. A portion of a new plant, which will treat the sewage by settlement and single contact beds, has been put under construction and will be sufficient to handle practically all of the dry weather flow. On the subject of capacity (life) and efficiency of contact beds, Mr. Charles F. Wike, M. Inst., C. E., City Surveyor of Sheffield, states in his report:

The records of capacity gagings of the experimental contact beds have been made systematically for several years, and show that, if carefully worked, reduction of capacity will only proceed at a slow rate, ensuring a length of life which brings such beds within economical limits. To keep them thoroughly efficient, and to give suitable periods of rest when there is a falling off in purification, it is essential that regular and systematic analyses of the effluents should be made. It is also necessary to have periodic gagings of capacity in order to avoid undue accumulations of organisms or organic solids. As far as possible suspended solids should be reduced in quantity by suitable settlement before the sewage is applied to the beds.

A LARGE LUMBER RAFT, said to contain 11,000,000 ft. of lumber, arrived at San Francisco on Aug. 23, having been towed there from Astoria, Ore. It is reported that the raft was 60 ft. wide, 740 ft. long and drew 24 ft. of water; also, that the trip from Astoria was made in seven days.

THE AMOUNT OF FREE WATER IN THE EARTH'S crust is discussed at length by Mr. Myron L. Fuller in Water Supply and Irrigation Paper No. 160 of the U. S. Geological Survey. After reviewing the estimates made by previous theorizers on this subject, and discussing in detail the various points involved, Mr. Fuller states his conclusion (in substance) as follows:

The various factors affecting the computation of the volume of underground water may be tabulated as follows:

Rocks.	Thickness, ft.	Porosity, %	Saturation, factor, %	Volume occupied by water, %
Sandstone	1,040	15.0	37	5.25
Shale	1,300	4.0		1.48
Limestone	260	5.0		1.75
Crystalline rocks	15,375	.2	50	1
	17,975	....	...	....

Average per cent. of rock occupied by water, 0.52.

On the basis of these facts the total free water held in the earth's crust would be equivalent to a uniform sheet over the entire surface with a depth of little less than 100 ft. (96 ft.), which is only about 1/75 of the amount postulated by Delesse, 1/28 of that of Slichter, 1/16 to ¼ of that of Chamberlin and Salisbury, and 3/7 of that of Van Hise. It is recognized that locally, where the sediments are very porous and of considerable thickness, several times the amount of water estimated may exist in unconsolidated deposits, or in the stratified rocks alone. There is a general tendency, however, to overestimate the amount of water in the ground owing to the impression of great volume which a large well often conveys, the fact that a large area is drained being frequently overlooked. The writer's studies of the conditions in deep wells in the United States lead him to the belief that the average amount of water present in the earth is probably under, rather than over, the amount estimated.

A 4,000,000-FT. TIMBER RAFT has left Astoria, Ore., for San Diego, Cal. The raft is the property of the Benson Log & Lumber Co., and is expected to make the trip of 1,500 miles or so in some half-dozen days, reaching San Diego about Sept. 15.

PERSONALS.

Mr. W. B. Pason has been appointed Assistant City Engineer of Wilkes-Barre, Pa.

Mr. William S. Bacot, Engineer of the Consolidated Water Co., of Utica, N. Y., has resigned.

Mr. F. T. Rembert has been elected Secretary and Treasurer of the Texas & Gulf Ry., to succeed Mr. G. A. Rogers, resigned.

Mr. D. T. Forbes, Superintendent of the Victoria Division of the Southern Pacific Ry., has resigned to become Superintendent of the Colorado & Southern Ry.

Mr. Henry W. Klare, formerly Chief Engineer of the Virginia Bridge & Iron Co., has accepted the position of General Manager of the Bartlett Steel Co., of Joplin, Mo.

Mr. E. P. Bracken has been appointed Superintendent of the Sheridan Division of the Chicago, Bunrlington & Quincy Ry., to succeed Mr. F. D. Weidenhamer, resigned.

Mr. C. J. Carroll has been appointed Assistant Chief Engineer of the Mexican International R. R., with office at Ciudad Porfirio Diaz, to succeed Mr. Rankin Johnson, resigned.

Mr. R. G. Manning, Assoc. M. Am. Soc. C. E., Engineer of the Toledo plant of the American Bridge Co., has been appointed Engineer of the Ambridge plant of the same company.

Mr. F. du P. Thomson has closed his engineering office in the Fidelity Bldg., Buffalo, N. Y., and will give his entire time to the interest of the Wheeling Mold & Foundry Co., of Wheeling, W. Va.

Mr. Geo. W. Craig, Assoc. M. Am. Soc. C. E., has resigned as Assistant City Engineer of Omaha, Neb., and has opened offices at 422 the Bee Building, [illegible] city, to engage in engineering and contracting.

Mr. T. R. Aiken, formerly Assistant to the Vice-President of the Republic Iron & Steel Co., has been made General Manager of the plants of that company, succeeding Mr. Chas. R. Hart, resigned to become President of the Inland Steel Co.

Mr. H. C. Phillips, who has been in charge of the [illegible]-building for the Atchison, Topeka & Santa Fe [illegible] in the Eureka District of Northern California, has been appointed Chief Engineer of the Coast Lines of the company to succeed Mr. C. A. Morse, transferred.

Mr. W. B. Poland, who has been in charge of the location and construction of the Alaska Central R[illegible] since Jan. 1, 1905, has resigned. This is the new road which is being built from Seward, on Resurrection Bay, to the Tanana River in the interior of Alaska, a distance of about 500 miles.

Mr. P. M. Corry has been appointed Assistant Engineer in charge of lines and grades in the field at Grand Central Terminal of the New York Central & Hudson River R. R. Mr. E. D. Sabine has been appointed Assistant Engineer in charge of inspectors in the field, Grand Central Terminal, to succeed Mr. Corry.

Mr. M. M. March has been appointed Assistant Engineer of the Cotton Belt & Northern Ry., with headquarters at Texarkana, Ark. He will be in charge of the reconstruction of the road between Texarkana and Mt. Pleasant, Texas, and the ballast work on the Fort Worth Branch between Mt. Pleasant and Greenville.

Mr. Howard E. Troutman, formerly Manager of the Chicago office of the Buckeye Engine Co., has accepted the position of Sales Manager of the Corliss and High Speed Engine Department of the Atlas Engine Works, of Indianapolis. Mr. J. M. Broucher, of the Brownell Co., of Dayton, Ohio, becomes Assistant Sales Manager.

Mr. Chas. L. Parmelee has accepted the position of Consulting Engineer with J. G. White & Co., Inc., and will superintend their work in California and Arizona. He will thus be in charge of the work of completing the Laguna Dam, which this company is building across the Colorado River for the U. S. Reclamation Service.

Mr. J. S. Barlow, formerly Assistant Engineer in charge of construction of the Arizona Eastern R. R., in the Gila Cañon, Ariz., has been placed in charge of the field parties on construction in the Yaqui River and Alamos Branch Lines of the Cananea, Yaqui River & Pacific R. R., with headquarters at Corral, Sonora, Mex. He succeeds Mr. O. C. Thompson, transferred.

Obituary.

F. J. Korteck, a contractor of San Francisco, died in that city Aug. 31, aged 45 years.

David Brown, a contractor of Brooklyn, N. Y., died of heart failure at his home in that city, Sept. 4; 73 years of age.

Paul Dickinson, President of the Paul Dickinson Iron & Locomotive Works, of Chicago, died at his home in Winnetka, Ill., Sept. 1. He was born at Milwaukee, Wis., in 1867, but was educated at Chicago, where he later entered the iron business.

ENGINEERING SOCIETIES.

COMING MEETINGS.

THE NEW ENGLAND WATER-WORKS ASSOCIATION.
Sept. 12-14. Annual Convention at Fabyan House, White Mountains, N. H. Secy., Willard Kent, Narragansett Pier, R. I.

AMERICAN PUBLIC WORKS ASSOCIATION.
Sept. 14-15. Second Annual Meeting at Atlanta, Ga. Secy., W. H. Flint, Chattanooga, Tenn.

LEAGUE OF AMERICAN MUNICIPALITIES.
Sept. 20 to 28. Annual convention at Chicago, Ill. Secy., Jno. MacVicar, Des Moines, Iowa.

AMERICAN SOCIETY OF MUNICIPAL IMPROVEMENTS.
Oct. 2 to 5. Annual Meeting at Birmingham, Ala. Secy., G. W. Tillson, Municipal Bldg., Brooklyn, N. Y.

AMERICAN ELECTROCHEMICAL SOCIETY.
Oct. 8-9. Annual meeting at New York City. Secy., S. S. Sadtler, 39 South 10th St., Philadelphia, Pa.

AMERICAN STREET AND INTERURBAN RAILWAY ASSOCIATION.
Oct. 15 to 19. Annual Convention at Columbus, Ohio. Secy., Bernard V. Swenson, 60 Wall St., New York City.

THE RAILWAY SIGNAL ASSOCIATION.
Oct. 16-18. Annual Meeting at Washington, D. C. Secy., H. S. Balliet, Grand Central Station, New York City.

APPALACHIAN ENGINEERING ASSOCIATION.—The quarterly meeting of the association will be held at Welch, W. Va., Sept. 14. The following day the plants of the U. S. Steel Co. will be inspected.

VERMONT ELECTRICAL ASSOCIATION.—The fifth annual meeting of the association was held at St. Johnsbury, Vt., Sept. 12-13. Wednesday, Sept. 12, was spent at Lake Park, where a shore dinner was served. Thursday was devoted to business and the reading of a paper by Mr. Francis W. Wilcox on "A Review of Recent Improvements in Incandescent Lamps."

ENGINEERING LITERATURE

Supplement to ENGINEERING NEWS. New York, September 13, 1906.

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REVIEWS.

Electrical Books.

Reviewed by Henry H. Norris.*

STRAY CURRENTS FROM ELECTRIC RAILWAYS.—By Carl Michalke. Translated and Edited by Otis Allen Kenyon. New York: McGraw Publishing Co. Cloth; 5½ × 8¼ ins.; pp. 101; 34 illustrations in the text. $1.50, net.

DESIGNS FOR SMALL DYNAMOS AND MOTORS.—By Cecil P. Poole, M. Am. Inst. E. E. New York: McGraw Publishing Co. Cloth; 6 × 9½ ins.; pp. 186; 231 figures in the text. $2, net.

A great deal has been written on the subject of corrosion produced by the return currents from electric railways and many suggestions have been made as to the manner in which the trouble may be avoided. The literature of the subject is widely scattered and it is therefore very convenient to have in compact form the essential features. The author of this work has made a careful, practical and analytical study of electrolysis which the translator alleges to be "a disease peculiar to America." With characteristic German thoroughness the work of many authorities has been scrutinized and the results abstracted and arranged for convenient reference. The translator has increased the value of the original by the addition of notes from American practice and by an ample bibliography of German and American articles.

For convenience of review the book may be divided into three parts. Of these the first consists of mathematical studies of the electrical conditions by which corrosion of pipes and cable coverings is caused. A number of cases are analyzed, and the general principles by which the amount of leakage current may be calculated are given and illustrated algebraically. The second section, of a single chapter, deals with the subject of corrosion and the conditions under which it may take place. Following this are important chapters on the practical application of the theory to the detection and prevention of destructive electrolysis. A chapter on measurement gives directions for such tests as will indicate the existence of dangerous stray currents. Following is a discussion of the methods in use for reducing or preventing the trouble. It is encouraging to note that in Europe the effects of stray currents have not been serious in spite of predictions to the contrary. It is probable that the matter is of more importance in this country if one is to judge by the amount of attention given to it. All engineers and owners who are concerned with underground pipes of any kind must be conversant with the subject, for if the conditions are favorable much damage may be caused by these stray currents. Hence the value of a concise and yet thorough work like that of Dr. Michalke. The book is provided with a complete index and a brief table of contents, and in addition there is a convenient table of the symbols used in the formulas. While some of the mathematical proofs require a knowledge of advanced calculus, the more complicated of these are in foot-notes, and the conclusions may be accepted without going deeply into detail. The practical parts of the treatment may be followed by a reader with a slight mathematical training.

*Professor of Electrical Engineering, Cornell University, Ithaca, N. Y.

Mr. Poole's book comprises a number of articles which have been previously printed in the American Electrician and part of which are included in the author's "Electrical Designs." Several of the best designs are by Mr. J. C. Brocksmit. The range of the book is considerable, including both direct and alternating current dynamos and motors of various sizes from a fan motor to a 75-light incandescent dynamo. The book is to form the first of a series of four, the three others covering "electrical instruments, telephones and similar light constructions; heavier electrical apparatus other than dynamos and stationary motors; and gas engines and other machinery not involving electrical construction." In the present volume the descriptions will be intelligible to any person who is already somewhat familiar with the construction of such machines. It will, presumably, be of interest to young men who are of a practical turn of mind and who desire to apply their talent for mechanical construction in a useful direction. There is undoubtedly much to be learned by this process. It is well, however, to caution the reader that the building of an electrical machine requires a rather high order of skill, and that more or less disappointment will usually result when the product is compared with the commercial article of the same capacity. The facilities of an excellent machine shop must also be available. The book contains only the elements of each machine, the smaller details being left to the ingenuity of the constructor, who is advised to copy them from successful commercial machines or to buy them from a manufacturer. As a matter of fact, it would probably be cheaper to purchase the complete machines outright, but in this case the pleasure and the educational value of building and assembling the parts would be lost.

The general plan of each chapter, comprising one design, is to describe the actual details in the form of working directions. The underlying principles and reasons for the various steps are not given. This is a fault which might have been avoided, and which greatly decreases the educational value of the work. Fairly complete working drawings are given, and these will be clear to any one familiar with ordinary shop practice. The book contains an assortment of designs comprising details of construction of a number of small dynamos and motors, including modern, self-starting, single-phase induction types as well as the older forms. A series of three sizes of combined alternating and direct-current machines should interest teachers and pupils in manual training high schools, where such equipment would be useful for laboratory practice. The sizes given, 1, 2 and 4 KW., are rather small for a college laboratory. The concluding design is one of a rectifier, such as could be used for charging storage batteries from an alternating current circuit.

As a whole, the book will be useful for reference, and the designs appear to be carefully worked out. Some of the motor frames are of rather crude form but, on the other hand, many of the machines are of modern appearance. The printing and illustrations are of good quality.

STADIA DIAGRAM.—Corrected for C = 1.20 ft. Horizontal Correction and Difference of Elevation. Designed by B. W. Homans, Civil Engineer, New York City. New York: Charles Hart (36 Vesey St.). Sheet, 22½ × 28½ ins. $2.50.

This is an instrument rather than a diagram. It comprises a chart, about 9 × 20 ins., for each of the two quantities desired, and an alidade for each diagram, all printed on a large sheet of heavy cardboard. Near one corner of either diagram is a center to which the alidade is to be fixed by a pin, and swung over the diagram to the desired angular reading of elevation. The coordinates are rod-reading, taken along the alidade, and angular distance, laid off around the diagram on enlarged angular scale. Curves of equal value laid off on each diagram determine the desired quantity, horizontal correction or elevation.

Steel Bridge Specifications.

GENERAL SPECIFICATIONS FOR STEEL RAILROAD BRIDGES AND STRUCTURES.—With a Section Making Them Applicable to Highway Bridges and Buildings. Prepared under the Direction of Virgil G. Bogue, Civil Engineer, by Albert W. Buel, Associate on Bridges, 15 William St., New York. New York: The Engineering News Publishing Co. Paper; 6 × 8¾ ins.; pp. 61; diagrams and tables. 50 cts., net.

There is more than is usually found of consistent effort in this new set of specifications to arrange the individual clauses in definite groups, where Material or Stress clauses shall not be mixed in with Workmanship clauses. Mr. Buel's specification is also noteworthy for considerably higher stresses than are generally allowed. Thus, 24,000 lbs. per sq. in. in bearing on the projected area of rivets is allowed, and 24,000 lbs. bending fiber stress in pins, against 15,000 and 18,000 lbs. allowed by Cooper; but Buel increases his live-loads by an "impact" allowance, whereas Cooper does not. Similarly, Buel allows twice Cooper's loading of expansion rollers. The workmanship specifications given by Buel are strict.

One of the vital requisites in a specification, absolute clearness of expression and prevention of all ambiguity, is brought to mind by several places where imperfections of language are confusing. The columns of viaduct towers, for example,

> will have a batter transversely not less than 6 vertical to 1 horizontal for single track, and not less than 8 vertical to 1 horizontal for double track,

which means, we presume, that they *shall* have a batter of not *more* than 6 units vertical to 1 unit horizontal for single track, etc. Again, a most vital clause, that prescribing the "impact allowance," is not free from ambiguity. Again, it is provided that swing spans "shall be cambered an amount equal to the maximum calculated deflection," which may refer to the end deflection or to the deflection of the half span. This book of specifications is good enough to promise living into a second edition; if it does, we trust that these ambiguous points may be cleared, and that the author will add an auxiliary section which will cover shop-buildings, as the present section on "Highway Bridges and Buildings" fails to do so.

NOTES ON METALLURGICAL MILL CONSTRUCTION.—Edited by W. R. Ingalls. New York: The Engineering and Mining Journal. Cloth; 6¼ × 9½ ins.; pp. 256; 40 figures in the text and 1 plate. $2.

The material comprising this book consists of a series of articles relating to the design, construction and operation of metallurgical mills. These articles have appeared, chiefly during the last three years, in "The Engineering and Mining Journal," and several of them are abstracts of papers presented before engineering societies. Although the book cannot be called a general treatise on metallurgical mills, owing to the absence of articles on many important matters, the subjects taken up are discussed by engineers of broad experience, and are particularly interesting by reason of several of the engineers being from different continents. The articles are classified under seven headings, as follows: Brickwork and Concrete; Building Construction; Ore-Crushing Machinery; Dryers and Drying; Conveyors and Elevators; Disposal of Tailings; and Miscellaneous. Much of the matter classified as "Miscellaneous" consists of editorials on metallurgical mill construction reprinted from the journal named.

Industrial Furnaces.

Reviewed by Joseph W. Richards.*

INDUSTRIAL FURNACES AND METHODS OF CONTROL.—By Emilio Damour, Ingenieur Civil des Mines. Authorized Translation with Additions by A. L. J. Queneau, Consulting Engineer, The New Jersey Zinc Co. New York and London: The Engineering & Mining Journal. Cloth; 6 × 9½ ins.; pp. 317; 88 figures in the text, and 24 tables. $4.

Just half of this work is a translation of Professor Damour's "Le Chauffage Industriêl et les Fours à Gas"; the other half, comprising Pyrometry, Gas Analysis, Chimneys, and Design of Regenerative Furnaces, is original work of the modest but able translator, while chapters on Calorimetry and the Ultimate Analysis of Fuels, have been written by Myrich N. Bolles. A salutatory by Prof. Henry M. Howes completes the list of sponsors for the work.

Regarding the translation proper, it is not done into classical English, but it is always comprehensible. Its merits and demerits are therefore practically those of the original. Its subject matter comprises the calculation of heat of combustion of fuels, temperatures of combustion under varying conditions, heat distribution and thermal efficiency of furnaces, application of these principles to gas producers and various kinds of recuperative and regenerative furnaces. In this discussion, water gas is practically omitted, the short note on this subject by the translator being insufficient to supply this very serious deficiency. The definitions in this part of the work are in a number of cases either antiquated or erroneous; molecular and atomic weights are confused in several instances, recuperation and regeneration are used interchangeably, the heat utilized in a furnace is said to include all heat lost from the hearth, both by radiation and conduction (in which case, increasing the radiation losses would increase the efficiency of a furnace!)

Aside from some fundamental mistakes, such as those mentioned above, the detail of the subject matter is highly commendable—patiently and painstakingly worked out, and the methods of analysis evolved are of great industrial value. In this exposition of principles we would prefer to see kilograms and cubic meters used instead of grams and liters, as being more practical units, and we believe that the use of the molecular volume, 22.32 liters, as the unit of volume in the gaseous calculations, is sure to mystify many practical men, who cannot comprehend its use as skilled chemists can. Nevertheless, the principle is correct and correctly used, but it makes the subject much harder to master, for a large circle of practical men who cannot be called ignorant because unacquainted with this chemical principle.

The question of pyrometry is written by Mr. Queneau himself, in a very satisfactory manner, and contains many details of management, such as reveal the experience of the writer, and will be highly instructive to the reader. The latest forms of practical pyrometers are described, analysed and soundly criticised. A similar judgment may be expressed regarding the chapter on gas analysis, also by Mr. Queneau.

The subject of calorimetry is equally well handled by Mr. Bolles, who writes also from much practical experience in this line. His advice will be found reliable and precise, and of value to anyone attempting this field of laboratory work. The following chapter on elementary fuel analysis, also by Mr. Bolles, is too short to cover the ground satisfactorily. It is good as far as it goes.

The remainder of the book (Part III.) dealing with the design of chimneys, regenerators, recuperators, valves, ports, hearths, and the use of powdered fuel, is strongly written. There is hardly any other book in English which collects these data and sets forth the principles so clearly or so completely. Some parts of this subject might well have been more extensively treated, however; the design of the hearth is handled far too briefly, considering its great importance, and no data are given concerning its dimensions for given capacities of furnace.

Altogether, the work presents to the English reader the slightly antiquated but still valuable work of Damour, with its shortcomings mostly uncorrected in the translation; in addition, the valuable, first-hand contributions of a well-trained specialist, Mr. Queneau, upon furnace design, pyrometry, and the use of powdered fuel; and a well-written chapter on calorimetry. Such composes a work very instructive to an engineering student; valuable, but rather difficult to master, to the practical engineer; interesting, but not without many shortcomings, in the eyes of the theoretical metallurgist. To any of these, it is well worth its cost.

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23 MURRAY and 27 WARREN STREETS, NEW YORK

LESSONS ON SANITATION.—A Handbook for Students. Arranged on the Principle of Question and Answer, Forming a Complete Course of Study on the Subject. By John W. Harrison, M. R. San. Inst. London, England: Charles Griffin & Co., Ltd. Philadelphia, Pa.: J. B. Lippincott Co. Cloth; 5¼ × 8 ins.; pp. 175; 53 illustrations in the text. $1.50 net.

These "lessons" present, in the catechism form so much favored by certain instructors and learners, the essential elementary features of public sanitation as conceived and practiced in England. After taking up water supply, ventilation, plumbing, certain phases of building construction, infectious diseases, disinfectants, and the various divisions of the work of health inspectors, a considerable amount of space is devoted to statutes and regulations governing factories, tenements, food and milk supply, and the like. The book, as a whole, is better suited to enable a would-be inspector to pass an examination than to ground him in the principles of health-protective work. But there is doubtless a legitimate demand for just such books; and if every food and nuisance inspector had at his hand the substance of the information in this volume, and conducted his work accordingly, the health conditions of municipalities would be greatly improved within a decade.

GENERAL SPECIFICATIONS FOR STEEL RAILROAD BRIDGES AND VIADUCTS.—New and Revised Edition. By Theodore Cooper, Consulting Engineer. New York: The Engineering News Publishing Co. Paper; 7 × 9½ ins.; pp. 36. 50 cts., net.

The 1906 edition of this standard bridge specification differs from its immediate predecessor in nearly 50 clauses where changes or additions have been made.

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Instructors in Civil Engineering, Massachusetts Institute of Technology

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ADVANCE CIRCULARS WILL BE READY SHORTLY

JOHN WILEY & SONS, 43 and 45 East 19th St., New York City

A Compendium of Practical Mathematics.

HANDBOOK OF MATHEMATICS.—For Engineers and Engineering Students. By J. Claudel. From the Seventh French Edition, Translated and Edited by Ottis Allen Kenyon. New York: McGraw Publishing Co. [illegible]; 6 × 9½ ins.; pp. 708; 422 figures in the text. $3.50, net.

A useful compendium of the common or "practical" branches of mathematics, well written and well translated, is offered in this work. Within the covers of a single volume is given the whole range lying between simplest arithmetic and differential and integral calculus. It is a singular thing, in a time when more than ever before the specialized monograph is in demand, to see an effort in the opposite direction in so vast a field as that of mathematics. In the present case the effort is worthy and, as far as may be expected, successful. The demands which individual users will place on such a work must necessarily be so varied that many omissions will be felt. Thus, for our own part we regret the absence of a section on imaginaries and complex quantities—so useful in various dynamical and electrical studies—and the omission of a table of integrals, the resource of many a man suddenly called upon to solve a problem involving an integration. An index to the book would also prove very convenient in many an instance, and a list of the tables which are now hidden in the pages of the volume—from tables of prime numbers to a mortality table—would repay the trouble of preparing it. But the individual criticism in the case of a work of this scope must necessarily remain individual.

UNDERGROUND-WATER PAPERS, 1906.—By Myron L. Fuller, Geologist in Charge. Water-Supply and Irrigation Paper No. 160, U. S. Geological Survey. Charles D. Walcott, Director. Washington, D. C.: Pub. Doc. Paper; 6 × 9 ins.; pp. 104; one plate and four text illustrations.

Eleven papers on subjects relating to underground water are included in this pamphlet. Of these, six are by Mr. Fuller; of the six, the one of most general interest is probably a discussion of the significance of the term "artesian." The discussion includes a review of the history of the word; the results of a canvass among American geologists and hydrologists as to their preferences in the matter of dropping the term "artesian" entirely, on the one hand, or, on the other, giving it a meaning of (1) flowing wells, (2) any well in which water rises above the level at which it is found, and (3) any deep well. The arguments for and against the various uses are summarized and discussed. The paper closes with a series of proposed definitions. The conclusion of the whole matter is that in the nomenclature of wells the artesian or hydrostatic principle should be recognized, rather than such an element as depth.

Of the other papers by Mr. Fuller, a brief one discusses and presents a series of symbols for representing wells and springs on maps, and another presents an interesting review of estimates of the amount of free water in the earth's crust, followed by Mr. Fuller's own conclusions, which latter put the amount at a lower figure than has been reached by those who have preceded Mr. Fuller in this line of speculation. A particularly notable paper included in the collection is one by Mr. R. B. Dole, on "Use of Fluorescein in the Study of Underground Waters." Considering its brevity, Mr. Dole's paper gives a remarkably comprehensive review of the general subject named, including a historical outline and mathematical discussion, some tabular matter, the methods to be employed in using this system of underground flows, and, finally, a bibliography of the subject.

CONGRÈS INTERNATIONAL DE TRAMWAYS ET DE CHEMINS DE FER D'INTERET LOCAL.—Milan, Sept. 17-21, 1906. Brussels, Belgium: International Tramways and Light Railways Union (General Secv., 6, Impasse du Parc). Paper; 8¼ × 13¼ ins.; pp. 649.

This volume contains answers to questions sent out by the Congress to European roads early in the present year. It gives a general idea of the construction of these roads, their operation and the advantages or disadvantages obtaining from the apparatus used.

JOHN WILEY & SONS'
SCIENTIFIC PUBLICATIONS

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By Henry Solon Graves, M.A., Director of the Forest School, Yale University. 8vo, xiv + 458 pages. Cloth, $4.00. *September*

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Including Natural Stone, Plain-concrete, and Reinforced-concrete Arches. For the use of Technical Schools, Engineers, and Computers in Designing Arches according to the Elastic Theory. By Malverd A. Howe, M. Am. Soc. C. E., Professor of Civil Engineering, Rose Polytechnic Institute. 8vo, x + 170 pages, many illustrations. Cloth, $2.50.

Occurrence and Properties of Clay and Its Distribution in the United States
By Heinrich Ries, Ph.D., Assistant Professor of Economic Geology, Cornell University. 8vo, cloth, xvi + 490 pages, 65 figures and 44 half-tones. $5.00 net. *September*

Notes on Military Explosives
By Erasmus M. Weaver, Major, General Staff, U. S. Army. 8vo, cloth, v + 311 pages. $3.00 net. *September*

Manual of Wireless Telegraphy
By A. Frederick Collins. 12mo, x + 232 pages, 90 figures, and 1 communication chart. Cloth, $1.50 net; morocco, $2.00 net. *September*

Elements of Mechanical Drawing
By Alfred A. Titsworth, Professor of Civil Engineering and Graphics, Rutgers College. Oblong 8vo. *September*

Notes on Least Squares and Geodesy
By Charles L. Crandall, Professor of Civil Engineering, Cornell University. 8vo, cloth. *October 1st*

Problems in Surveying, Railroad Surveying, and Geodesy
With an Appendix on the Adjustment of the Engineer's Transit and Level. By Howard C. Ives, Asst. Professor of Railroad Engineering, Worcester Polytechnic Institute, and Harold E. Hilts, Instructor in Civil Engineering, University of Pennsylvania. 16mo, morocco, $1.50.

Principles and Practice of Surveying
A Text-Book for Engineering Colleges and a Work of Reference for Engineers. By Charles B. Breed and George L. Hosmer, Instructors in Civil Engineering in the Massachusetts Institute of Technology. 8vo, cloth. *September*

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The Construction and Equipment of Cranes.

KRANE (Construction and Equipment of Cranes).—Ein Handbuch für Bureau, Betrieb und Studium. By Anton Böttcher. With the collaboration of G. Frasch. Munich and Berlin, Germany: R Oldenbourg. Cloth; 6½ × 9½ ins.; pp. 500; 492 figures in the text, and 41 tables; 48 plates bound separately. 25 marks; American price, $10.

The wide use and development of cranes, which in the last dozen or twenty years have made crane construction a highly specialized field, render a comprehensive work on the subject very appropriate. The author of the present work, who claims considerable experience in crane work, has aimed to provide at once a comprehensive and concise treatise: comprehensive, in order to do justice to the multiplicity of forms and types of cranes, and the large number of structural and mechanical problems and devices which they involve; concise, in order to make the book conveniently suitable for use as a reference manual and to keep its price within reasonable limits. Considering this twofold aim the work is to be commended. The book is somewhat similar in general character to the "Hebezeuge" of Prof. A. Ernst, but is in much more condensed form.

It contains sections on the general arrangement or types of cranes, on the principal constructive details of cranes, on the design of plate and lattice girders for cranes, and on the motive power of cranes; besides chapters on the statics and dynamics involved in the completed machine and in its elements, and a section giving descriptions and illustrations of eleven actual cranes of various types and capacities, illustrated by 48 folding plates in a separate volume from the text. A valuable additional feature is an appendix containing a large amount of pertinent information, including tables of steel shapes and moments of inertia, and various standard specifications for steel construction, electrical safety appliances, and rating and testing electrical machinery.

EXPERIMENTS ON THE STRENGTH OF TREATED TIMBER.—By W. Kendrick Hatt, Ph.D., Civil Engineer, Forest Service. Circular 39, Forest Service, U. S. Department of Agriculture. Gifford Pinchot, Forester. Washington, D. C.: Pub. Doc. Paper; 6 × 9 ins.; pp. 31; 2 text figures and 13 tables.

The testing plant, treatments employed before testing, and the methods and results of testing various materials, are described in this pamphlet. The number and character of the tests are as follows: 184 green loblolly pine 6 × 8-in. hewed ties from Texas; 20 seasoned loblolly pine 6 × 8-in. hewed ties from Texas; 165 seasoned loblolly pine 6 × 8-in. hewed ties from Texas; 97 partially seasoned western yellow pine sawed bridge ties from New Mexico and Arizona; 10 seasoned loblolly pine 7 × 9-in. sawed ties from eastern Texas.

THE SAN FRANCISCO EARTHQUAKE AND FIRE.—A Brief History of the Disaster. A Presentation of Facts and Resulting Phenomena, with Special Reference to the Efficiency of Building Materials. Lessons of the Disaster. By A. L. A. Himmelwright, C. E., M. Am. Soc. C. E. New York: The Roebling Construction Co. Cloth; 11 × 9½ ins.; pp. 270; numerous illustrations. $5.

Based on a three weeks' energetic and careful study of the San Francisco ruins, this work of Mr. Himmelwright is a valuable fund of material concerning the effects of the great earthquake and fire on city buildings. The author's business interest led him to restrict his study to the field of city buildings, and indeed this is an integral and sharply individualized portion of the San Francisco "results." By combining trained and close observation with judicious criticism of facts, Mr. Himmelwright has produced an excellent monograph. While it is published by a construction company interested in certain proprietary types of concrete construction which at San Francisco were in competition with other types, yet the influence of this fact, it seems, may be readily discounted by the reader if one desired to reject most of the text, the large collection of excellent photographic views would make the book worth while. The views cover a wider field of detail on the San Francisco buildings than any we have yet seen, and the text itself merits high commendation.

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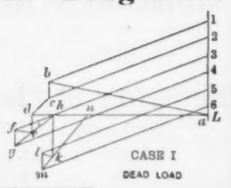

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